GUIDELINES FOR GEOTECHNICAL SITE INVESTIGATIONS IN RHODE ISLAND

FINAL REPORT

RIDOT Study - 0103
March 30, 2005

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16. Abstract

An accurate and professional geotechnical site investigation is critical for the proper design and construction of structures. There is a continued trend towards the use of specialized in situ tests such as the cone penetration test, pressuremeter, and cross-hole shear wave velocity tests. These specialized in situ tests can supplement and/or replace a standard drilling and sampling investigation.

This manual provides RIDOT engineers with clear and concise guidelines for understanding, planning, conducting, and evaluating geotechnical site investigations in Rhode Island. The advantages and disadvantages of both standard and specialized site investigation techniques are presented. Correlations between *in situ* test results and engineering properties of soils are also presented. The guidelines focus on the specific soil conditions found in Rhode Island as well as the size and scope of geotechnical investigations that are necessary for different types of structures commonly dealt with by RIDOT. This manual is not to be used as a specification for RIDOT site investigations.

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DISCLAIMER

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Chapter 1: Introduction

Chapter 2: Description of Soil Types in Rhode Island

Chapter 3: General Site Investigation Guidelines

Chapter 4: Standard Penetration Test

Chapter 5: Other *In Situ* Testing Methods

1. Introduction

A cost effective and thorough geotechnical site investigation (including field and laboratory testing) is critical for the proper design and construction of roads and structures. The primary purpose of a site investigation is to identify and characterize the types of soils and rock present at a site and the location of the groundwater table. In many cases the focus is on locating unsuitable soils such as fill, organics, and contaminated soils. Loose, saturated deposits of sands and silts are also a concern because of their potential for liquefaction during earthquakes.

The Rhode Island Department of Transportation (RIDOT) is involved in many types of projects that require geotechnical site investigations. Bridge piers and embankments such as the new (2004) Washington Bridge and Providence River Bridge are supported on both shallow footings and deep foundations. Many of these structures also include retaining walls with their own foundation systems. These projects require detailed and useful information about the soil conditions that will be encountered. Information is also needed about groundwater conditions for excavations during construction and foundation subgrade preparation.

Geotechnical site investigations in Rhode Island typically involve borings with standard penetration tests. The blow counts from the standard penetration test are used to estimate engineering properties such as density, strength, and compressibility. Representative soil samples from both SPT and test pits are used to classify the soil, and if fine-grained soils are encountered undisturbed samples using thin-walled samplers are obtained for laboratory strength and consolidation tests. This information is used to estimate the strength, bearing capacity, and settlement behavior of foundations. More specialized *in situ* tests such as the cone penetration test, pressuremeter, and field vane test are also occasionally performed in Rhode Island to supplement traditional site investigations. Cone penetration tests and cross hole seismic tests were performed as part of a seismic evaluation of the Washington Bridge located on the Seekonk River. Pressuremeter tests were performed to evaluate the viability of drilled shafts for a ramp at Interstate Route 95 near the Providence Place Mall. The increased use of these techniques brings challenges to RIDOT engineers, who must ensure that geotechnical site investigations are cost effective and result in an accurate assessment of soil properties for foundation design.

1.1 Relationship Between RIDOT, Contractors, and Consultants

Successful planning and execution of a geotechnical site investigation involves coordination between RIDOT personnel, engineering consultants (the "prime" and geotechnical subconsultant), and contractors. Initial planning and design for most RIDOT projects begins with the RIDOT Bridge and/or Highway Engineering sections and the prime design consultant. The prime consultant develops a Design Study Report or other preliminary assessment that presents recommendations for improvement, replacement, or construction of bridge structures and highway alignments. The need for a geotechnical foundation evaluation and new or supplemental borings is based upon the findings of the preliminary studies. If substructure rehabilitation or new construction is proposed, then a geotechnical engineering consultant, usually subcontracted to the prime design consultant, is added to the design team.

Design locations for proposed substructures or highway alignments are usually selected by or prior to the 10% design submission. Typically the geotechnical subsurface exploration program should be performed early in the design, during the 10% design phase. Boring locations and soil and rock sampling and testing programs should be designed to address all structural, foundation design, and construction issues and to anticipate subsurface conditions. However, design development or initial findings might result in the need for additional borings to be performed later in the design development. Occasionally borings are performed during project construction. Such construction phase borings are conducted to verify conditions at specific substructure foundation locations (e.g. through the center of drilled shafts) or as part of geotechnical instrumentation installation programs designed to monitor groundwater and soil behavior during construction preloading, excavation, or foundation installations.

A RIDOT geotechnical site investigation will generally consist of three phases:

1. Planning and contracting of the subsurface boring and sampling program and associated laboratory testing of soil and rock samples;

2. Conducting the drilling, downhole sampling and/or testing, and well installations; conducting laboratory testing of soil or rock samples;
3. Performing geotechnical analyses and developing and reporting subsurface findings and foundation design and construction recommendations.

RIDOT, the prime design consultant, the geotechnical consultant, and the drilling contractor will all have certain responsibilities and tasks during these phases. Table 1.1 (a) to (c) summarizes the various actions associated with each phase of the investigation and illustrates the functions of each party.

1.2 The Geotechnical Findings and Recommendations Report

The principal purpose of the geotechnical report is to present and communicate the geotechnical engineering consultant's opinion as to: 1) feasible options for support of highway and bridge structures; 2) specific recommendations for the most suitable foundation support; 3) recommendations for necessary construction installations and procedures; and 4) identification of geotechnical issues or difficulties which may impact construction, and appropriate solutions for those issues.

While the geotechnical report content and format will vary with project size and intent, all RIDOT project geotechnical reports should contain certain basic essential information, including:

- A description of the proposed structure or roadway construction;
- A site location plan and a Subsurface Exploration Location Plan showing "as-built" boring locations and ground surface elevations;
- Summaries of the subsurface exploration data, including boring logs, a subsurface soil and bedrock profile, laboratory and in situ testing results;
- Description of the subsurface soils, rock, and groundwater findings and observations;
- Appropriate bearing capacity, settlement, stability and seismic analyses;
- Recommendations of specific soil and bedrock engineering properties to be used for design;

- Discussion of feasible foundation support options, to include advantages and disadvantages;
- Specific recommendations for the best or most feasible foundation support option;
- Discussion of the need for earth support, dewatering, or other construction procedures and specific recommendations for the design of such systems;
- Discussion of subsurface conditions which may be encountered during construction, and presenting recommendations for solution of anticipated problems.
- Appropriate references and calculations.

RIDOT typically requires submission of both a draft and final geotechnical report. The final report is generated subsequent to RIDOT review and comment on the draft report. However, early and ongoing communication among RIDOT, the structural design consultant and the geotechnical engineering consultant will benefit development of the structural design and will likely identify potential construction difficulties at an early stage. For large, complex projects, geotechnical reports specific to individual structures might be necessary. Occasionally, project size, complexity or other factors might require that both a geotechnical data report and a subsequent interpretive and recommendations geotechnical report are to be submitted.

RIDOT engineering staff also use the geotechnical report during review of the final (30% to PS&E) design submissions. The intent of this review is to ensure that the geotechnical engineering recommendations have been incorporated in the design plans and specifications. Typically and as a minimum, the boring locations should be shown in plan view on a base plan sheet with elevation contours. Copies of the boring logs should be included on plan sheets or in the contract book. The contract documents should also make reference to availability of the geotechnical report for review by the construction contractor.

A more comprehensive treatment of the geotechnical findings report, appropriate analyses and calculations, design recommendations for specific soil and rock types, and development of foundation options and recommendations will be addressed in future RI-

DOT guidelines. However, the reader is referenced to the FHWA publication "Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications", FHWA-PD-97-002, dated October 1985.

1.3 Objectives and Scope of this Manual

The objective of this manual is to provide RIDOT engineers with clear and concise guidelines for understanding, planning, conducting, and evaluating geotechnical site investigations in Rhode Island. **This manual is not to be used as a specification for RIDOT site investigations.** The advantages and disadvantages of both standard and specialized site investigation techniques are presented. Correlations between *in situ* test results and engineering properties of soils are also presented. The guidelines focus on the specific soil conditions found in Rhode Island as well as the size and scope of geotechnical investigations that are necessary for different types of structures commonly dealt with by RIDOT.

Chapter 2 of this manual presents a description of the geologic history of Rhode Island as it relates to the distribution of soil types in the state. Local bedrock types and geological stratigraphy commonly encountered along major RIDOT alignments are described, and descriptions of local soil and rock are presented.

Chapter 3 presents general guidelines for planning and conducting geotechnical subsurface investigations for the variety of structures, embankments, excavations and subsurface facilities associated with RIDOT projects. Recommendations for type of drilling and the number, spacing and depth of borings are presented. Drilling and soil and rock sampling equipment and methods available and commonly used by local drilling contractors are described. Recommended split spoon and undisturbed soil sampling practices are discussed. This chapter also discusses the typical bid and pay items to be considered when developing the subsurface program.

Chapter 4 describes the Standard Penetration Test (SPT) method of sampling subsurface soils. In addition to providing a sample which is generally representative of the type and gradation of soils present at the sampling depth, the SPT penetration resistance, called the "blow count", is the initial information used to determine the suitability of that strata for foundation bearing, and to evaluate other likely soil behavior under proposed structure or response to construction excavation. The chapter discusses the specified equipment and procedures used for the SPT, factors that affect the resulting blow count data, and correlations with engineering properties of soils.

Chapter 5 presents a description of other *in situ* testing methods including the cone penetration test, pressuremeter, field vane test, and cross hole and down hole seismic tests. These methods are not commonly employed locally, but have been occasionally used by RIDOT to provide data for evaluating soil and rock properties or behavior for specific project needs.

Table 1.1(a) Specific responsibilities of different parties for planning and conducting geotechnical site investigations in Rhode Island.

Planning

RIDOT	Prime Design Consultant	Geotechnical Consultant	Drilling Contractor
Review and approval of proposed scope of geotechnical services, scope of drilling and soils testing, and estimate of costs.	Provide structure locations and alignments for anticipated structure configurations and loads. Provide project base mapping, with ground surface elevation contours. Develop and submit manhour/cost estimates. Develop traffic maintenance and protection plan.	Develop proposed scope and locations of borings and test pits, schedule of depths, soil and rock sampling, boring location plan, and specific drilling notes. Prepare scope and manhour/cost estimate for geotechnical consultant services. Prepare estimate of drilling costs for review by RIDOT and design team.	
Direct formal advertising or open solicitation for drilling services based on cost estimates provided by the consultants. Review and approval of contractor bids.	Coordinate and assist RIDOT and geotechnical consultant with development and solicitation and advertising (if necessary) for drilling and testing program.	Prepare bid package (drilling scope, boring location plan, notes, and RIDOT drilling specifications) for solicitation of drilling contractor's bids.	Review plans, scope of drilling, work site access, etc.
		Review Contractor's bids and provide recommendations for selection of driller.	Develop and submit itemized cost bid to mobilize and perform the borings, provide excavator, and conduct test pits.
Maintainence of RIDOT standard drilling specifications, in-house project records for local geology and soil conditions, and Standards and Guides of the MUTCD.			

Table 1.1(b) Specific responsibilities of different parties for planning and conducting geotechnical site investigations in Rhode Island.

Exploration

RIDOT	Prime Design Consultant	Geotechnical Consultant	Drilling Contractor
Coordinate with prime and geotechnical consultants as needed to schedule boring program and provide assistance or approval of change in scope during the drilling.	Coordinate with geotechnical consultant and drilling contractor as needed.	Layout boring and test pit locations for use by drilling contractor and for utility clearance.	Contact RI DigSafe and other subsurface utility owners.
	Survey ground elevation and location of completed borings. Locations should be tied to State Plane coordinate system (feet) and standard datum. Water borings require coordination and surveying during the drilling, while barges are on station.		Provide appropriate equipment for drilling. Provide barge for water borings, maintain tide boards or facilities for monitoring water depth and mudline elevation. Provide and maintain traffic control equipment and devices. Arrange for the use of local potable water during drilling.
RIDOT may provide inspection services during drilling.	Provide observation services during drilling if geotechnical consultant is not part of the project team. Prime consultant is ultimately responsible for completed final boring logs.	Provide on-site observation and decision making during drilling (as RIDOT's representative). Conduct field tests in conjunction with driller's operation. Collect, maintain, schedule and deliver selected soil and/or rock samples for appropriate laboratory testing (some of this may be done by the driller).	Perform the drilling, soil and rock sampling in accordance with the contract scope. Prepare and maintain field boring logs during drilling. Collect, label, and maintain soil and rock samples during drilling, and transport to temporary or permanent storage or disposal. Prepare and submit final boring logs.

Table 1.1(c) Specific responsibilities of different parties for planning and conducting geotechnical site investigations in Rhode Island.

Evaluation and Reporting of Findings

RIDOT	Prime Design Consultant	Geotechnical Consultant	Drilling Contractor
		Review boring logs and soil or rock samples.	Deliver "typed" hard-copies of the drilling logs to the prime or geotechnical consultant as per contractual agreement.
Review draft and final reports, including • Geotechnical data report • Geotechnical interpretative report • Geotechnical design summary report RIDOT will determine the reporting format to be used for the particular project.	Deliver geotechnical reports and final boring logs to RIDOT and coordinate with geotechnical consultant for preparation and response to draft report review comments.	Prepare draft and final geotechnical findings and recommendations reports, including • Engineering properties of site soils and bedrock • Foundation alternatives • A recommendation of the most feasible foundation option • Appropriate construction, excavation, earth support, dewatering, etc.	Provide temporary storage and/or delivery of soil and rock samples for testing or long-term storage.
Coordinate inclusion of geotechnical design and construction recommendations into design plan and specifications submissions. Review and consider construction phase services proposal by Geotechnical consultant.	Incorporate findings of geotechnical, foundation, and earth support recommendations into final design plans and specifications. Develop plans and specifications for construction. Include geotechnical issues for Contractor's attention during construction.	May be tasked to provide cost estimates for various foundation alternatives. Recommend construction phase services for consideration by RIDOT.	

2. Description of Soil Types in Rhode Island

2.1 Introduction

The types and distribution of soils in Rhode Island vary throughout the state and present many challenges to geotechnical engineers for the design and construction of foundations, retaining walls, excavations, and embankments. This chapter presents a brief overview of the glacial geology of Rhode Island and its role in soil deposition within the region, knowledge of which can provide insight into the soil types at a proposed site. Common soil types will be described according to their geologic origin, including glacial till, outwash deposits, and recent fills. Problem soils such as organics, loose saturated sands and silts, fills, and boulders are also discussed.

Most of RIDOT's projects involve alignments of state roads and bridges. Hence, the majority of RIDOT's experience is along the roadway corridors through Providence, the interstate highways, north-south along the west bay coastal area, north-south along the major east bay roadway corridors and east-west major bridge alignments over the bay and local tributary estuaries. RIDOT has relatively less experience and familiarity with subsurface conditions in the northwest and western areas of the State.

2.2 Geology of Rhode Island

The geologic history of Rhode Island extends to millions of years, however of most concern for understanding the current soil types is the last glaciation during the Wisconsinan period that ended approximately 10,000 years ago (Quinn 1976). During this period, all of Rhode Island was covered with glacial ice several thousand feet thick, and the sea level was much lower than present levels. The Wisconsinan ice sheet reached as far south in this area as Long Island, Block Island, and Martha's Vineyard, as shown in Figure 2.1. Another advance and subsequent retreat of the ice sheet created the end moraines along Charlestown in southern Rhode Island and Cape Cod in Massachusetts.

As the ice sheet traveled southward, it scraped away the existing soil and rock down to the bedrock. The soil and rock were carried in the ice until it melted, at which time the soil was re-deposited as a well-graded mixture of gravel, sand, silt, and clay called glacial till. Till is found today directly overlying the bedrock throughout the state.

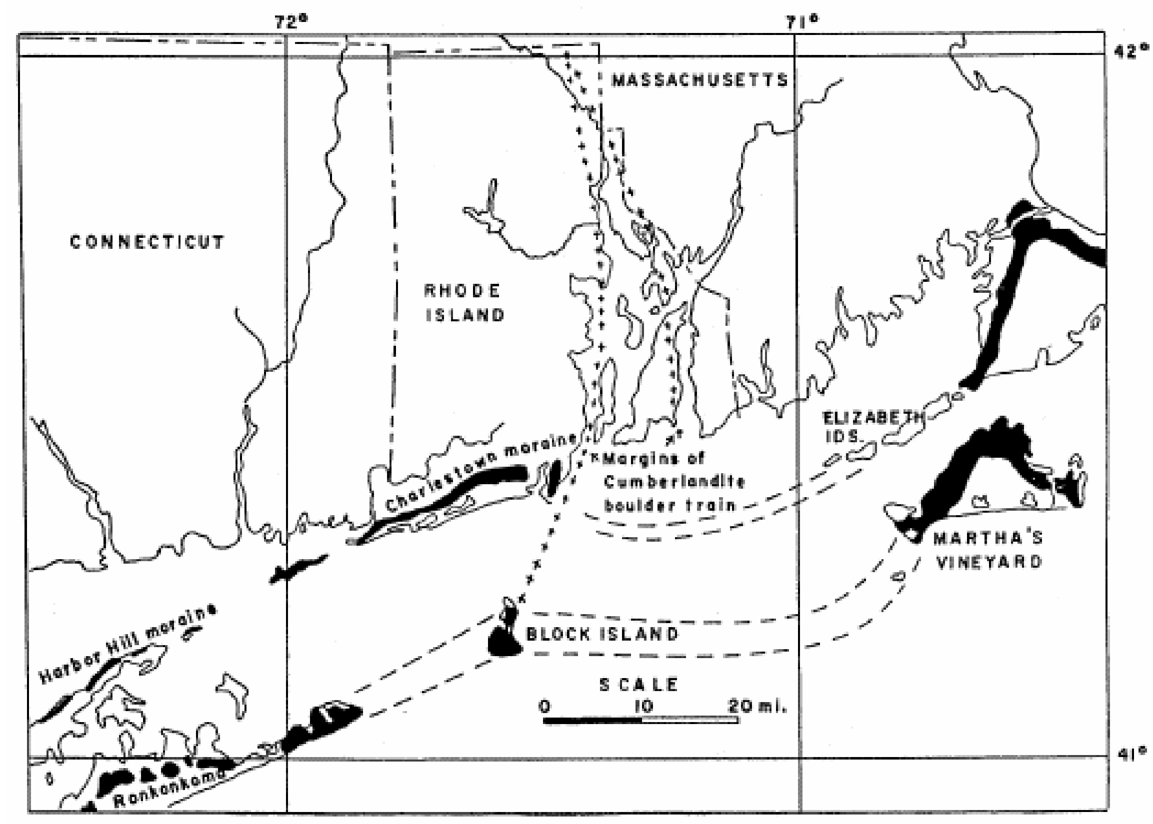
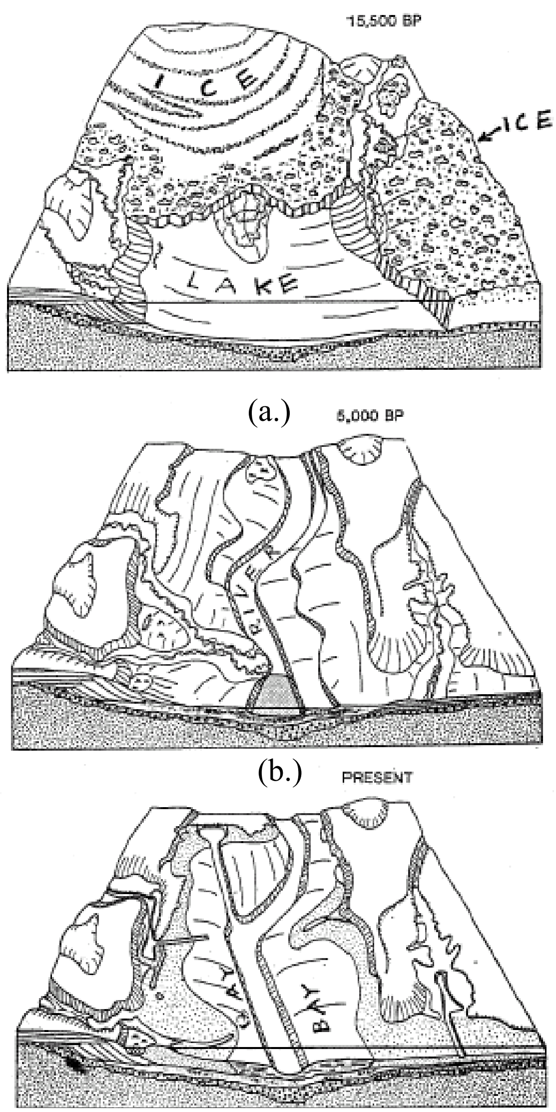


Figure 2.1. Two bands of terminal moraines that illustrate the movement of the Wisconsinan ice sheet (Murray 1988).

Narragansett Bay was formed 15,000 to 20,000 years ago as the melting ice formed a lake that covered an area larger than the current Bay. This progression is shown in Figure 2.2. The southern boundary of the lake consisted of a natural levee that was formed earlier during the advancing ice sheet. The water level in the lake reached a height approximately 30 feet above the present sea level (Murray 1988). This is illustrated in Figure 2.2 (a). The majority of the outwash soils, such as sands and inorganic silts, found in and around Narragansett Bay was deposited during this period over glacial till. An outwash deposit refers to a soil that is transported by glacial meltwater and deposited in a region beyond the terminal edge of the glacier (Ritter et al. 1995). Outwash materials tend to be poorly graded sands and gravels having rounded particles. In Rhode Island, however, outwash deposits can consist of thick sequences of silts overlain by gravelly sands. Underlying downtown Providence, for example, varved silt deposits (called the Providence Silts) are commonly found with thicknesses ranging from 50 to

150+ feet. Thick stacks of silts or sands also occur south of the city in parts of Warwick, Cranston, Davisville (North Kingstown), and Narragansett.

When the natural levee broke, the lake drained and portions of the lacustrine outwash deposits within the Bay were exposed and were subsequently eroded by rivers and streams, as shown in Figures 2.2 (b) and 2.2 (c). As ice melted throughout the world, the sea level rose to its present level, leaving additional deposits of marine sands and silts both in the Bay and in the areas surrounding the Bay. Due to this complex geologic history, the location and extent of the outwash deposits in Rhode Island are extremely variable.



(c.)

Figure 2.2. The geologic development of Narragansett Bay (Murray 1988).

2.3 Bedrock

Bedrock type and occurrence (i.e. depth to the rock surface) is of major importance during a subsurface investigation when overlying soils are known or anticipated to be unsuitable for structural foundation support, or where structure, utility trenches or roadway excavations will require significant and expensive rock removal. Knowledge of the type of rock and quality (degree of weathering, fracture and joint frequency) will enable the design engineer to evaluate rock strength and bearing capacity, suitability of the upper rock zone for rock socketing, or relative resistance to mechanical methods of excavation. Some local rock types exhibit weathered or intensely fractured zones that may extend significantly (>10 to 20 feet) below the rock surface. Alternatively, some rock types possess bedding or foliation, which may be potential planes of weakness if oriented at high angles relative to proposed structural loads or result in unstable rockcut faces along roadway alignments.

For detailed illustrations, mapping, and descriptions of specific rock types and their distribution in the state, the reader is referred to the USGS state quadrangle maps, the State Bedrock Geology map, "Rhode Island, The Last Billion Years" (Murray 1988) and "Rhode Island Geology for the Non-Geologist" (Quinn 1976).

Figure 2.3 shows a simplified map of the distribution of major bedrock types underlying Rhode Island and adjoining areas of Connecticut and southeastern Massachusetts (Murray 1988). In general, the bedrock of Rhode Island may be considered in terms of two major groupings. The first group consists of crystalline granitic and gneissic rocks and various other metamorphic rocks underlying the northern and western upland portions of the state, the south shore area (Westerly to Narragansett) and the east bay area from Tiverton to Little Compton. On the figure these consist of pre-Cambrian (PC) granites in the northern part of the state and on the east underlying Tiverton, Little Compton and southeastern Massachusetts, the Scituate Granite mass in central Rhode Island, the Narragansett Pier Granite along the south shore, and gneisses and undifferentiated metamorphosed sedimentary and volcanic rocks of the Hope Valley terrane in the southwest and underlying eastern Connecticut.

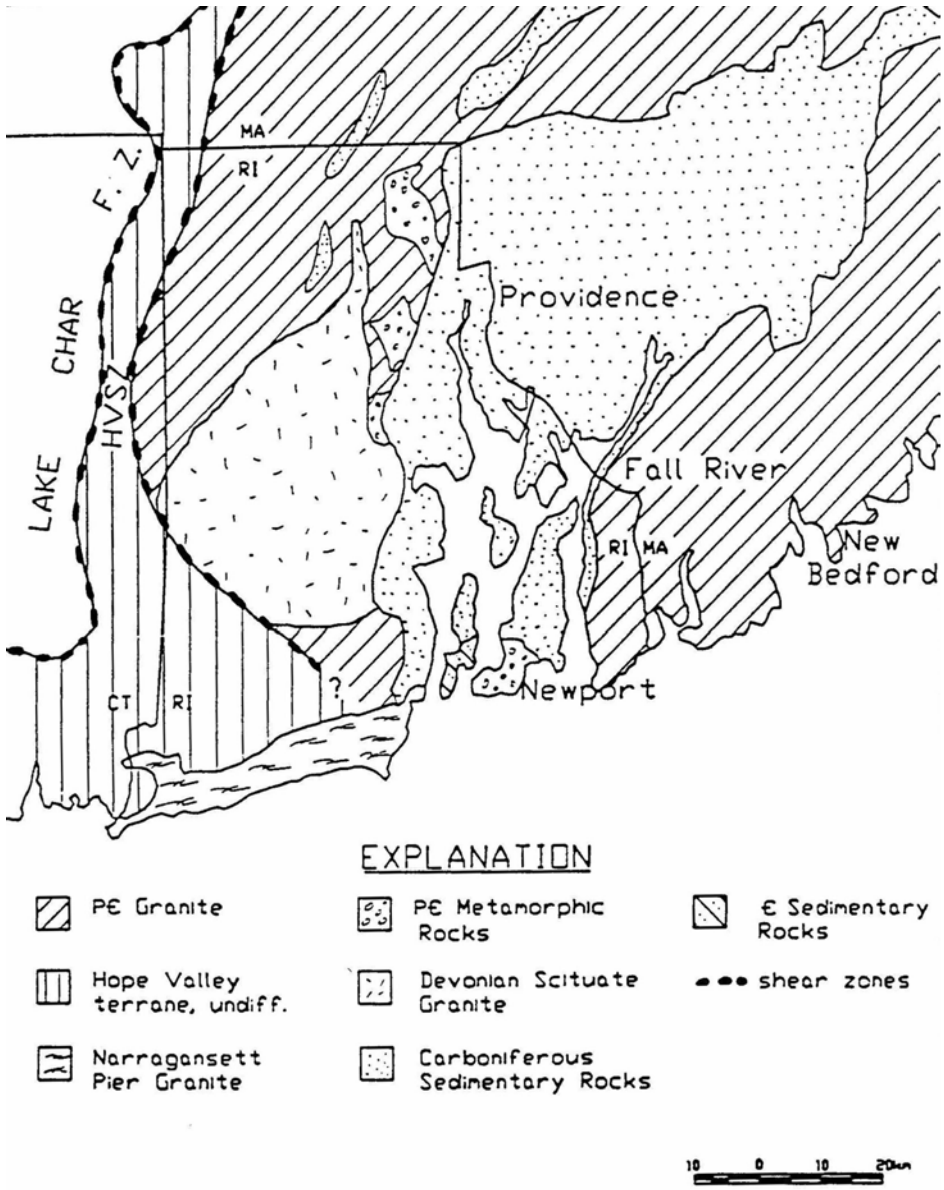


Figure 2.3. Simplified bedrock geology of Rhode Island. PC – Precambrian, C – Cambrian, F.Z. – Fault Zone (Murray 1988).

These rocks have been relatively resistant to weathering and erosion. Although subjected to glacial scour, they remain the framework for upland terrain in the north and western part of Rhode Island and control the orientation of local stream/river drainage patterns and gradients. Bedrock surfaces tend to be relatively shallow or locally exposed (naturally or by road cuts and gravel borrow operations), except where pre-glacial drainage has cut deep valleys in the bedrock surface. Examples of these rocks are most notable along west bay road cuts on Rtes I-295, I-146 and Rte 7, the Narragansett shoreline, and along Rtes 24 and 77 in Tiverton and Little Compton.

The second major bedrock group in Rhode Island is the sedimentary and meta-sedimentary rocks of the Narragansett Basin Formation, locally known as the Rhode Island Formation. The Narragansett Basin Formation occurs as a broad band, roughly 6 to 10 miles wide, and occupies the lowland area extending from southeastern Massachusetts, through the East Providence-Barrington-Warren-Bristol area, Providence, and west shore coastal areas to Narragansett. These rocks underlie Narragansett Bay and comprise large parts of the bay islands: Prudence, Conanicut Island (Jamestown), and Aquidnick Island (Newport/Middletown/Portsmouth). The constituent members of the formation include sandstones, shales, graphitic shale, conglomerates, and local coal seams (Portsmouth and Garden City in Cranston). However, at any locale, the rock should be expected to show great stratigraphic variability both laterally and with depth. By comparison to the first rock group, the Narragansett Basin Formation rocks tend to be less resistant to erosion and glacial scour. Rock surface elevations can be highly variable within relatively short distances.

2.3.1 Deep Bedrock Valleys

It is standard practice when investigating subsurface conditions at major structures to extend borings to bedrock and obtain core samples of the rock. This often requires borings on the order of 50 to 100 feet; however there are several areas in the state with very deep bedrock elevations due to the presence of pre-glacial river valleys that were likely scoured, deepened, and widened during glacial ice advance episodes. The result is bedrock surfaces on the order of 200 ft and greater below ground surface. These valleys were subsequently filled with glacial deposits of till, stratified outwash sands, gravels and silt, subsequent recent alluvial and organic soil deposits. Although the exact alignment and course of these pre-glacial bedrock valleys are not fully known, past RIDOT projects have encountered deep bedrock in the following areas:

- *Providence, underlying the Woonasquatucket River*: Sites include the Providence Place Mall, Farmer's Market, and rail yard. Depths to rock are approximately 200 to 220 feet. The rock consists of interbedded shale, graphitic shale, sandstone, and siltstone of the Narragansett Basin Formation. The apparent buried valley is bound by Exchange Street on the south and Capitol Hill and Smith Street to the north.

- *Providence, upper Narragansett Bay and Seekonk River entry into upper bay:* The depths to rock are approximately 100 to 125 feet deep along the west shore of the mouth of the Providence River, and 70 to 140 feet deep (below mudline) across the Seekonk River. The rock consists of Narragansett Basin shales, siltstone, sandstone, and conglomerates.
- *Providence-Cranston line, Cranston Street and Route 10 Viaduct*: Rock surfaces are up to 180 feet deep. The rock type is uncertain.
- *Narragansett/South Kingstown, Middlebridge over the Narrow River*: Rock surfaces are greater than 180 to 200 feet below the mudline. The rock type is uncertain.
- *Sakonnet River, Rte 24 over the Sakonnett River Bridge*: In some locations, rock surfaces are greater than 300 feet below the mudline. The rock type is likely shales and sandstones of the Narragansett Bay Formation, however, granites are found locally along the eastern shore.

At the Woonasquatucket River and upper bay sites, the soils consist of fill underlain by organic estuarine silt, a lower thin granular alluvium, and a thick (60 to 90 feet) deposit of the inorganic Providence silt. Till, where present, underlies the inorganic silt, and varies in thickness from approximately 10 to 20 feet.

The soils at the Route 10 Viaduct, Middlebridge, and Sakonnett River sites are believed to consist principally of thick glacial outwash sands, rather than silt.

One major impact of these bedrock surface depths upon planning subsurface exploration programs is that blanket requirements for bedrock coring may not be practical locally and are certainly expensive. Where deep foundations may be needed, alternatives to end-bearing foundations on rock should be assessed. In these cases, the site investigation program should also focus on defining the location and thickness of dense soils (e.g. glacial till, sand and gravel, etc.) that might serve as a bearing stratum.

2.4 Soil Types

As described above, the soils in Rhode Island are dominated by the movement of the Wisconsinan ice sheet that retreated approximately 15,000 years ago. The result of this history is that large areas of the state are underlain by glacial till and outwash deposits. Figure 2.4 shows the distribution of the four general geologic soil types found in the state: upland till plains, Narragansett till plains, end moraines, and outwash deposits. Glacial till is found over much of the eastern and western parts of the state. Outwash deposits are found near Providence, along the western edge of the Bay, and in southern Rhode Island. End moraines run east-west along the southern end of the state in Westerly and Charlestown and in Block Island. A fifth category of soil-types includes those that have been deposited since the end of the last glaciation. These soil-types may consist of alluvium, marine deposits, organic soils, and fill.

As an example, a typical soil profile is shown in Figure 2.5. This profile is from Fox Point in Providence and was developed as part of the re-alignment of Route I-195 through the city. The soils at this site consist of sand and gravel fill overlying organic silt, glacial outwash, till, and bedrock, and each of these soil types will be described in more detail in the following sections. The profile was interpreted from soil samples from the individual borings shown in the figure.

2.4.1 Glacial Till Deposits

Glacial till is material that was caught up in the ice as it advanced and scoured the land and bedrock. Till is typically encountered immediately overlying bedrock, as shown on Figure 2.5, and can be composed of transported material as well as local bedrock. The composition of the till in Rhode Island varies depending on the bedrock that the till is derived from. The upland till plains in the western and eastern portions of the state are derived from granite, schist, and gneiss rock. The Narragansett till plains around Narragansett Bay are derived from sedimentary rock, shale, sandstone, conglomerate, and, in some locations, coal (Murray 1988).

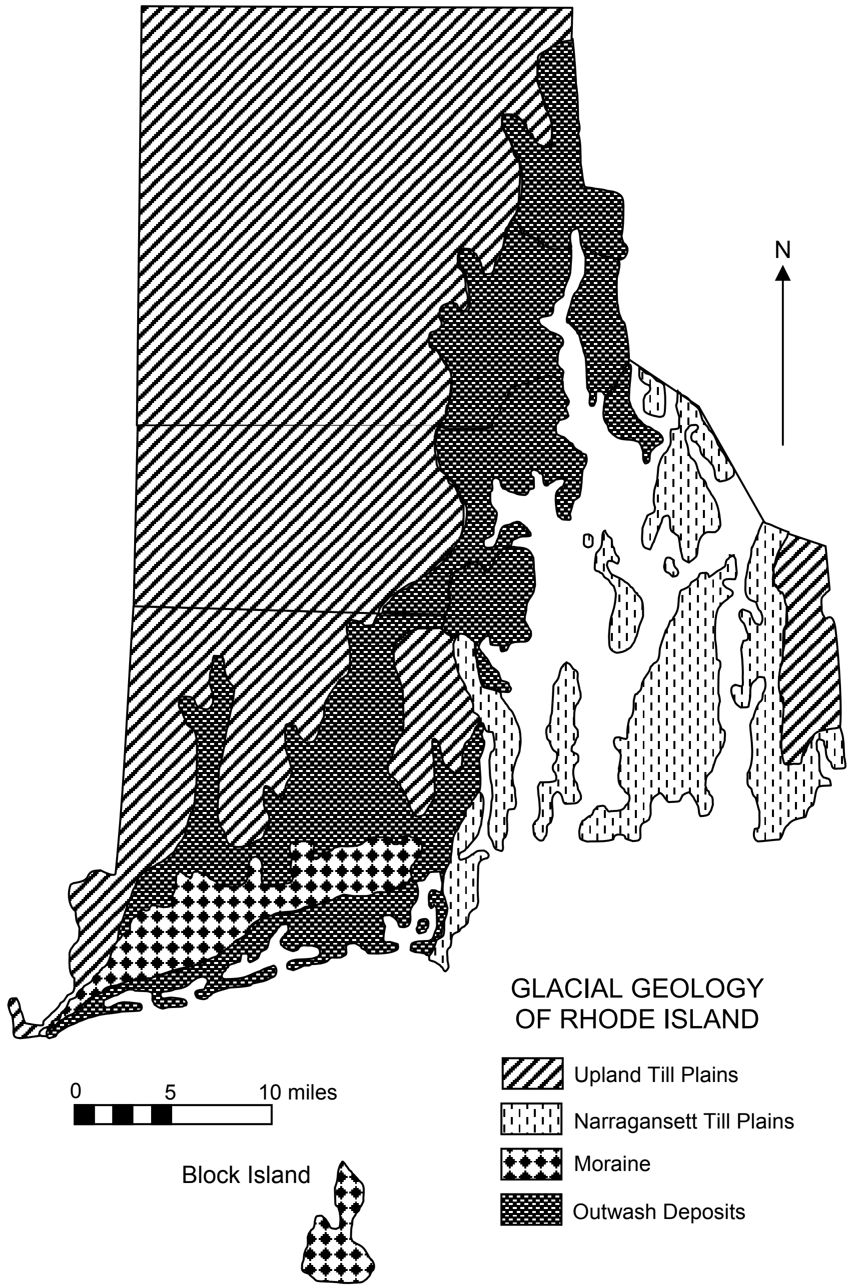


Figure 2.4. General location of soil deposits found in Rhode Island (after USDA 1981).

Till is characterized by a mass of a well-graded mix containing angular particles of a wide variety of grain sizes, ranging from clay sized particles to large boulders. Hard drilling and high standard penetration test (SPT) blow counts (i.e. greater than 50) can be an indication of the very dense and overconsolidated nature of till. Till is typically a good bearing layer for shallow or deep foundations.

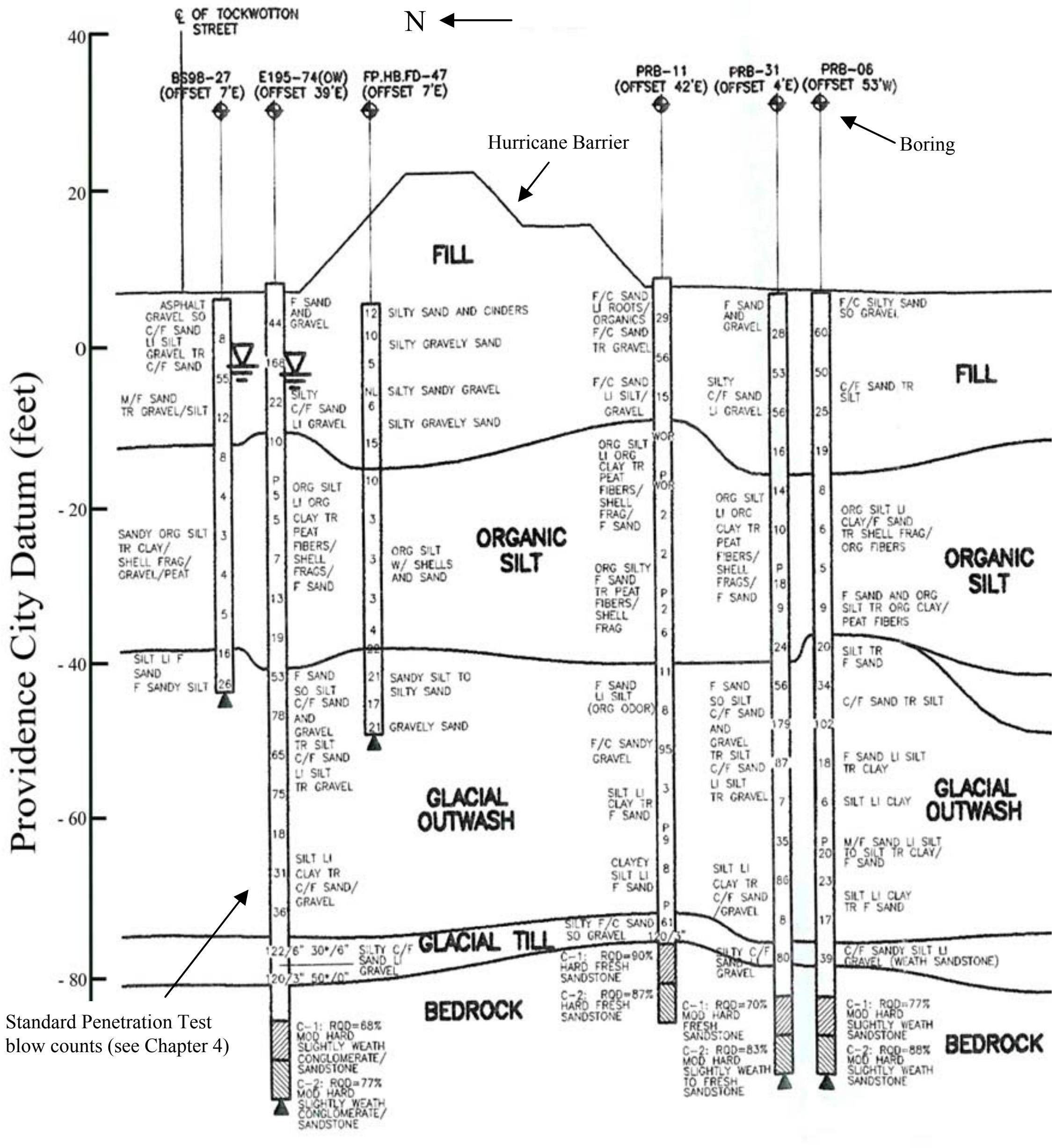


Figure 2.5. Example of a typical soil profile from Fox Point in Providence (GZA 1998).

2.4.2 Terminal Moraine Deposits

A terminal moraine deposit refers to a topographic feature formed by the accumulation of drift, or glacial, material that is deposited by ice (Ritter et al. 1995). The Charlestown and Block Island moraines are large deposits of boulders, sand, and silt that mark the southern limit of ice sheets in this area. The locations of these structures are shown on Figures 2.1 and 2.4.

2.4.3 Outwash Deposits

Outwash deposits refer to materials that have been transported by glacial melt-water and deposited either in a river (alluvial) or in a lake (lacustrine). These deposits in Rhode Island are generally located around Providence, along the western shore of Narragansett Bay, and along the southern coast of the state, as shown in Figure 2.4. Since outwash soils are deposited by the receding ice sheet, the outwash deposits are generally found overlying the till deposits, as shown in the typical soil profile in Figure 2.5.

The composition of the outwash deposits, however, can vary considerably from coarse-grained sand and gravel deposits to fine-grained inorganic silts. The coarse-grained deposits are a valuable source of Rhode Island's aggregate.

The inorganic Providence silts contain varying amounts of sand and clay-sized particles. The silts may also have horizontal bedding features such as varves that can affect drainage and other engineering properties. The density of the outwash deposits can range from very loose to very dense. Saturated silt is a problem in excavation activity due to easy disturbance by normal construction operations, and poses a challenge in obtaining high quality samples during a site investigation. The thickness of these deposits and the corresponding depth to bedrock also varies significantly from place to place.

Boulders can be encountered anywhere in soil profiles and can be very large (tens of feet in diameter). They are usually the product of ice rafting and are typically found in till and the coarse-grained outwash deposits. Boulders can cause problems during drilling, impact excavation efforts, and can complicate the interpretation of the suitability of the soils. A major concern during site investigations is that large boulders can be misinterpreted as bedrock, and experience is needed to ensure that bedrock has been reached.

2.4.4 Recent Deposits

Recent deposits (since the end of the last glaciation) may include a wide range of soils including organic deposits, alluvial deposits, marine deposits, and fill. The recent deposits are shown as organic silt and fill on the typical soil profile on Figure 2.5.

Freshwater organic deposits such as peat are formed from the accumulation and decomposition of plant matter. Peat can be identified by its fiberous-like structure. Peats can be problematic for foundation support since they undergo significant and continuous settlement if loaded by fill or structures.

Alluvial deposits (post-glacial recent deposits from streams and rivers) are often encountered overlying the glacial outwash soils. The alluvium is typically sandy material that ranges from clean to very silty. Sometimes the alluvium may contain roots or other organic material such as peat. The density of the alluvial soils can range from very loose to dense. These deposits can have a highly variable stratigraphy both vertically and laterally, and are generally suitable for construction purposes. However loose saturated deposits might be encountered that require improvement or replacement.

Marine deposits such as organic silts and clays may also be encountered overlying the outwash soils, particularly along coastal areas, as shown on Figure 2.4. The organic silts typically have medium to high plasticity, and can be very soft and compressible. Shell fragments identified within the organic silts can be an indication of marine deposits.

Most urban sites in Rhode Island contain a layer of surficial fill overlaying native soils. Fill refers to any material that is placed by man, and is generally used to raise grade for development. Two types of fill are considered in practice: structural and non-structural. Structural fill is free from debris, and is placed using engineering controls (i.e. compaction) making it suitable for foundation subgrades and backfill. Non-structural fill has not been placed using engineering controls and therefore may be in a loose condition or of suspect quality as a bearing material, and may contain debris. Fills containing organic debris such as trees, leaves, sawdust, etc., may be susceptible to settlement if the matter decomposes. Fills containing other types of debris such as construction debris, containers, pipes, or any other debris that can cause bridging, may compress or collapse upon loading, thereby causing settlements. Along the Providence shoreline, many deposits of organic silt are covered by such fill.

Fill is more easily identifiable if the fill contains indicators such as brick, concrete, or wood fragments. However, fill may be very difficult to identify in fills that consist purely of native soils. In these cases, the developmental history of the site may provide additional information in estimating the thickness or boundaries of any fill strata.

2.5 Groundwater Levels and Issues

Groundwater typically occurs at shallow depth in Rhode Island, often within 20 feet of the ground surface. Shallow groundwater would be expected in areas adjacent to wetlands, lowland areas adjacent to local ponds, rivers, and upper Narragansett Bay. In coastal areas and along local rivers and streams, groundwater levels can often be anticipated based upon surface water levels. However, in areas of the State characterized by uneven terrain, shallow tills and bedrock, or where subsurface soils may include significant thicknesses of impermeable soils, subsurface water levels may not exhibit uniform or "expected" levels.

The major design issues associated with groundwater occurrence and relative elevations will include the effective strengths of foundation soils, consolidation of compressible and organic soils, in-service hydrostatic loads on substructures, and long-term drainage along roadcuts or through embankments. Construction phase impacts are usually associated with drainage and dewatering of excavations, and the stability of wet or saturated subgrade soils when subjected to trafficking of construction equipment. Consequently it is important that the exploration program anticipate the issues associated with near-surface groundwater upon both design and construction.

Appropriately sited monitoring wells can be installed as the geotechnical borehole is completed, or in separate shallow borings. Such wells when included in the subsurface exploration program allow estimates of groundwater elevations which can be assumed to be representative of "stabilized" measurements. Significantly, the monitoring at specific locations can be repeated over time to estimate fluctuations seasonal fluctuations, or changes in groundwater level associated with specific precipitation events.

2.6 Soils Terminology and the Geologic and Geotechnical Perspectives

Previous sections have described the geologic origin and processes responsible for the distribution of subsurface soils and bedrock in the State. Geologic terms descriptive of soil type and depositional regimes (e.g. till, outwash deposits) or landforms (moraine) have been used. However, the engineering community has adopted terminology which describes soils by constituent particle size, and is intended to be suggestive of likely engineering behavior. This is the format which is used to describe subsurface soils on the typical boring log. The structural or geotechnical engineer tends to think of and group subsurface soil into separate strata characterized by 1) size of the major constituent particles of cohesionless, granular deposits; 2) distinguishing fine-grained silts and clay soils from the granular deposits; 3) identifying organic soils (fibrous to amorphous peat, organic silt, etc.); and 4) by the relative density or consistency of various layers or subsurface zones as may be indicated by SPT sampling data. We do, however, commonly use terms indicative of a soils geologic or cultural origin, eg: “fill”, “glacial till”, or “organic”, and associate certain physical properties and engineering behavior or consequences to them.

The structural and geotechnical engineer tend to take the local view by concentrating on the particulars within discrete boreholes. We attempt to reasonably predict or anticipate subsurface conditions at particular structure locations, based upon samples recovered from one or several 2-inch to 6-inch diameter holes. Consideration of the likely geologic depositional history can aid our understanding of local subsurface conditions. However, subsurface borings and test pits, in conjunction with appropriate sampling and testing, remain our best tools for determining the most suitable foundation system for soil and rock conditions.

3. General Site Investigation Guidelines

3.1 Introduction

Site investigations are performed to characterize the subsurface soils and to provide the necessary geotechnical properties for use in analyses and design. Several methods can be used to characterize the subsurface conditions at a given site including borings with standard penetration tests, cone penetration tests (CPT), and test pits. Drilling and sampling, whether performed using cased wash borings or auger borings, is the basic method available to RIDOT to evaluate the suitability of subsurface soils for foundation support and to identify conditions that may determine the need for a deep rather than shallow foundation.

Basic information from a drilling and sampling program is included on boring logs, which are the official record of drilling techniques used, testing and sampling data (blow counts, recovery, etc.), and observations made by the driller and the State's geotechnical representative. Boring logs record the soil and rock types encountered, groundwater conditions, equipment used, and in situ test results that are used to evaluate the suitability of the soils for the given project. The log describes soil and rock conditions found at a discrete point and multiple borings are typically necessary to adequately characterize the site for the intended construction. A sample boring log is shown in Figure 3.1.

Although the format of boring logs differs slightly depending on the company, all logs should contain the same basic information. An administrative section is typically in the upper left corner and contains information about the drilling company, project name, and location. The boring identifier is found in the upper right corner including the hole number, project number, and ground surface elevation. Other sections at the top of the log provide equipment and tooling data, groundwater observations, and dates of drilling.

The main body of the log contains information about how the hole was progressed and soil sampling data. This includes sampling depths, types of samples (e.g. split-spoon vs. fixed piston), SPT blow counts, and sample recovery. The remainder of the log is an interpretive description of the soil types encountered based on a visual characterization of the recovered samples and blow counts.

	PROJECT	REPORT OF BORING NO.
	Sample Boring Log	SHEET
		FILE NO.
		CHKD BY

BORING CO.		BORING LOCATION	Cranston, RI		
FOREMAN		GROUND SURFACE ELEV.	+12'	DATUM	
ENGINEER		DATE START	3/8/05	DATE END	3/8/05

SAMPLER: UNLESS OTHERWISE NOTED, SAMPLER CONSISTS OF A 2" SPLIT SPOON DRIVEN USING A 140 lb. HAMMER FALLING 30 IN

CASING: UNLESS OTHERWISE NOTED, CASING DRIVEN USING A 300 lb HAMMER FALLING 24 IN.

CASING SIZE: OTHER 3 3/4" HSA

GROUNDWATER READINGS				
DATE	TIME	WATER	CASING	STABILIZATION TIME
3/8/05		7.0 +/-		

DEPTH	FIELD TESTING	SAMPLE NO	SAMPLE PEN/./REC	SAMPLE DEPTH (FT)	SAMPLE BLOWS/6"	SAMPLE DESCRIPTION BURMISTER CLASSIFICATION	RK	STRATUM DESCRIPTION	
		1	24/11	0-2	15-5-5-4	Brown, fine Sand, trace silt, gravel			1
									2
									3
									4
5		2	24/15	4.5-5.5	2-2-3-4	Brown fine SAND, trace coarse sand, fine gravel			5
								FILL	6
									7
									8
		3	24/15	8.0-10.0	9-9-10-8	Brown fine SAND and Silt, trace brick		GWT	9
10									10
									11
									12
									13
		4	24/19	13.0-15.0	5-4-3-4	Brown SILT and fine Sand, trace wood and brick			14
15									15
									16
									17
									18
		5	24/21	18.0-20.0	6-12-14-	Gray sandy SILT			19
20					19				20
									21
									22
								STRATIFIED SAND	23
								AND SILT	24
25		UP1	12/12	24.0-25.0					25
		6	24/12	25.0-27.0	25-30-21-	Yellow Brown fine SAND and Silt			26
					18				27
									28
									29
30		UP2	24/24	29.0-31.0					30
									31
		7	24/19	31.0-33.0	7-8-10-11	Gray sandy SILT			32
									33
		UP3	21/18	34.0-35.8					34

GRANULAR SOILS BLOWS/FT	DENSITY	COHESIVE SOILS BLOWS/FT	DENSITY
0-4	VERY LOOSE	<2	VERY SOFT
4-10	LOOSE	2-4	SOFT
10-30	MEDIUM DENSE	4-8	M. STIFF
30-50	DENSE	8-15	STIFF
>50	VERY DENSE	15-30	V. STIFF
		>30	HARD

REMARKS:

UP = Fixed Piston

NOTES: 1) STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE AT TIMES AND UNDER CONDITIONS STATED, FLUCTUATIONS OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE

BORING NO. B-1

Figure 3.1. Typical boring log.

The boring log is also a document that becomes part of the construction contract. The contractor may use the log information to develop bid prices, plan methods, and choose equipment and the sequence of construction. During construction, the log provides a source of information for RIDOT construction staff to assess excavation depths, obstructions, and pile and sheeting penetration.

This chapter presents some general guidelines for conducting geotechnical site investigations including information about the following subjects: techniques for the design of shallow and deep foundations, types of borings, soil sampling, test pits, groundwater monitoring, classification of rock, and the economics of site investigations. A summary of selected publications that are relevant to this topic is also provided. These guidelines are directed specifically towards typical RIDOT construction projects involving deep and shallow bridge foundations, retaining walls, embankments, and associated earth excavation support.

3.2 Selected Publications on Site Investigations

There are numerous publications that provide recommendations and guidance for conducting geotechnical site investigations. The most relevant and current of these available publications are listed in Table 3.1. The 1997 Federal Highway Administration/National Highway Institute training course manual (Arman et al. 1997), in particular, is extremely thorough and covers subjects ranging from planning site visits to correlating the results of field tests to the engineering properties of soils. The 1988 AASHTO manual on subsurface investigations (American Association of State Highway and Transportation Officials 1988) includes detailed information on field mapping and the use and interpretation of geophysical methods.

3.3 General Guidelines

3.3.1 Types of Borings

Geotechnical site investigations usually require the drilling of boreholes to characterize the subsurface soils and to obtain samples for the determination of geotechnical properties required for analysis and design. Boreholes can be advanced using different methods, such as augering, rotary wash, and coring, depending on the subsurface conditions.

Auger borings are performed by attaching a drill bit to the leading section of an auger, called a flight, and drilling the auger into the ground. The soil is conveyed to the surface by the screwing action of the auger. The auger can either be a continuous flight (solid stem) or hollow (hollow stem auger), as shown in Figure 3.2. The continuous flight auger is used to drill in stiff, cohesive soils where the boring can remain open for the entire depth of the boring. As the auger is advanced more sections can be added until the desired depth is reached. Hollow stem augers are similar to the continuous flight auger except the center of the auger is hollow. A drill bit is used to plug the auger as the boring is advanced. When the boring reaches a desired depth, the bit is removed and sampling devices are lowered through the center of the augers.

Table 3.1. Selected Publications on Site Investigations.

Publication No.	Title
FHWA HI-97-021 NHI Course No. 13231	Training Course in Geotechnical and Foundation Engineering
FHWA HI-88-009 NHI Course No. 13212	Soils and Foundation Workshop
FHWA ED-88-053	Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications
USACE EM 1110-1-1804	Geotechnical Investigations
AFM 88-3 Chapter 7	Soils and Geology Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures)
AASHTO T 86 ASTM D 420	Guide for Investigating and Sampling Soil and Rock
AASHTO T 203 ASTM D 1452	Practice for Soil Investigation and Sampling by Auger Borings
NAVFAC DM 7.2	Foundations and Earth Structures

The hollow stem auger is generally preferred over the continuous flight auger since there is no need for the removal of the auger and the hollow stem auger acts as a temporary casing during sampling. Neither type of auger is usually used for sampling below the groundwater table due to the unbalanced water pressure acting against the soil at the bottom of the borehole, which leads to disturbance of the soil.

Figure 3.2. a) Solid stem auger versus hollow stem auger, b) Typical drilling configuration using hollow stem augers (DeJong and Bolanger 2000).

In order to minimize sample disturbance, rotary wash borings are typically used below the ground water table in silts, clays, and sands. A photograph of a rotary wash boring is shown in Figure 3.3. The borehole is either supported with casing or drilling mud to prevent the sides of the borehole from collapsing. When casing is used, it is driven or spun to the desired sample depth and the hole is cleaned with fluid and a rotary bit. A sampling device can then be lowered below the casing to obtain a sample. A head of water or drilling fluid should be maintained within the casing at or above the ground surface to reduce the stress relief to the soil caused by the drilling. This is critical for obtaining high quality samples for geotechnical testing.

The drilling fluid also facilitates in the removal of the soil cuttings to the surface. The drilling fluid is typically forced through the drill rods and out the sides of the bit to carry the cuttings up and out of the boring. A settling basin is attached to the pumping system on the ground surface to allow the coarse cuttings to settle out of the drill mud. The drilling fluid is then recirculated through the system. Observing the cuttings as they exit the borehole helps to identify changes in the soil conditions during drilling.

When boreholes must extend into weathered and unweathered rock formations, different drilling and sampling techniques are required. Intact rock samples for classification and laboratory testing are obtained by coring. For a more detailed discussion on drilling and sampling in rock, refer to ASTM D-2113, “Standard Practice for Diamond Core Drilling for Site Investigation.”

Figure 3.3. Typical Rotary Wash Boring System.

3.3.2 Soil Sampling

Soil samples must be obtained as part of any geotechnical site investigation to characterize subsurface conditions, determine geotechnical properties such as strength and compressibility, and corroborate the findings of *in situ* tests. A variety of techniques ranging from test pits to block sampling can be used depending on the use and desired quality of the samples. Important issues include the different methods used for obtaining soil samples, sample disturbance, and sampling intervals. Each of these is discussed below.

3.3.2.1 Sample Disturbance

When a sample is taken from the ground, there is some unavoidable amount of physical alteration of the soil structure both from sampling activities and from the removal of overburden stresses. Although the disturbance caused by the removal of overburden is difficult to avoid, the disturbance attributed to sampling is related to the type of soil and the sampling methods used. A sample that is considered to be "undisturbed" is one that is most representative of its *in situ* condition both in terms of its macro structure and mineralogical composition. By contrast a disturbed sample is one that may be representative of its mineralogical composition, but its macrostructure has been altered.

Undisturbed samples are critical for the accurate determination of engineering properties such as density, compressibility, permeability, and shear strength. In Rhode Island where the dominant fine-grained soils are inorganic and organic silts, however, obtaining undisturbed samples is extremely difficult. A recent study by Page (2004) showed that even careful drilling practices (weighted drilling fluid, fixed piston samplers, etc.) resulted in significant disturbance to samples of organic silts from Fox Point and inorganic silts from the old Farmer's market in Providence. This is important to recognize when evaluating laboratory test results such as pre-consolidation pressures and values of undrained shear strength.

Disturbed soil samples, like those obtained during a standard penetration test, can be very useful in evaluating the general stratigraphy of the site, performing soil classification, and obtaining index properties such as the grain size distribution and the Atterberg Limits. In addition, the SPT blow count provides very useful information and is used extensively in correlations of density and shear strength.

3.3.2.2 Sampling Methods

A variety of equipment and methods have been developed for the sampling of soils and rock. A complete discussion of these methods is given in the selected references shown in Table 3.1. Methods used for the sampling of soil include wash sampling, split spoon sampling, thin-walled (Shelby) tube sampling, stationary or hydraulic piston sampling, block sampling, and core barrel sampling for rock or very stiff to hard clays. However, the methods used in Rhode Island are generally limited to split spoon sampling (soil), Shelby tube sampling (soil), piston sampling (soil), and rotary core barrel sampling (rock). Each of these is described in more detail below.

Split Barrel Sampling (SPT) – The split barrel (split spoon) sampler is the most common tool for obtaining soil samples in geotechnical practice. It is used in the

Standard Penetration Test, which is described in Chapter 4. The sampler consists of two halves of pipe that are held together with a drive shoe at the bottom and a head assembly at the top, as shown in Figure 3.4.

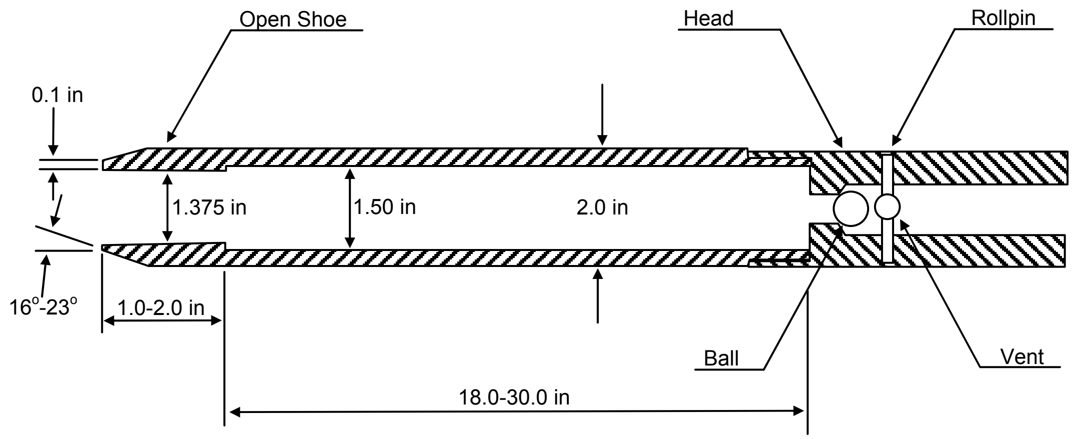


Figure 3.4. Split barrel sampler with dimensions (after ASTM D 1586).

The sampler is typically 18 to 30 inches (46 to 76 cm) in length with an inside diameter of 1.375 inches (35 mm). Once the split spoon containing the soil is recovered to the ground surface, it is decoupled from the drill rod, the drive shoe is removed, and the sample is split open. The sample contained within the split spoon is too disturbed from driving to obtain strength and compressibility data, however it is still representative of the ground conditions. A description of the sample is generally noted on the boring log, and sub samples of soil may be placed in containers for additional laboratory tests such as grain size analyses, water contents, and Atterberg Limits.

Thin-Walled Tube Sampling – Thin-walled (Shelby) tube sampling is generally used for obtaining undisturbed samples of peat, silt, or clay. The test method for thin-walled tube sampling is described in AASHTO T 207 and ASTM D 1587. The sampler dimensions consist of a 2-7/8 inch (73 mm) minimum inside diameter and a 3-inch (76 mm) maximum outside diameter and a corresponding area ratio of 9 percent. The area ratio is defined as the outside diameter of the sampler squared minus the inside diameter squared divided by the inside diameter squared. The sampler has a cutting edge that is fabricated for cutting a sample that is less than the inside diameter of the tube, thereby reducing side friction and disturbance on the sample. The tube is fixed to a head assembly that is attached to the end of the drill rod. The 36 inch long tube is pushed, not driven, into the soil at the bottom of the borehole typically 24 inches (600 mm). A ball valve located at the top of the sampler allows water and air to escape during penetration but helps to maintain the sample within the tube during removal. Immediately after penetration, a waiting period of at least 10 minutes is observed to allow for the sample to swell within the tube. The sampler is then rotated at least two complete revolutions to shear off the sample, and the sample is recovered to the surface for capping, labeling and transport to the laboratory.

Stationary Piston Sampling – A stationary piston sampler is particularly useful in soft peat, silt or clay where higher quality samples are needed than those obtained with a Shelby tube. Specific types of fixed piston samplers include the Osterberg sampler and the Guss sampler. Fixed piston sampling consists of a thin-walled sampling tube used in combination with a piston that is fixed at the soil surface at the bottom of the borehole, while the tube is pushed into the soil. The piston creates a vacuum at the top of the sample during penetration, thus reducing drag down and disturbance of the sample. The sampler tube generally has a 3-inch outside diameter and a wall thickness of 1.5 mm. The cutting edge is fabricated for cutting a sample that is 1/64-inch less than the inside diameter of the tube. The procedure includes lowering the sample tube with the piston positioned at the bottom of the tube. Once the piston reaches the bottom of the borehole, it is fixed and the tube is pushed around the piston and into the soil. After waiting at least 10 minutes, the tube is rotated to shear off the sample and recovered to the surface. At the surface, the vacuum created by the piston is broken, and the sample is capped, labeled, and transported to the laboratory.

3.3.2.3 Sampling Intervals

For split barrel sampling, a maximum interval of 5 feet is generally accepted in practice within Rhode Island. This is the most efficient depth interval because it corresponds to the length of a section of drill rod. However, it may be desirable to select a smaller interval to obtain a higher frequency of data, particularly within zones of influence below shallow foundations typically within the first 15’ below the proposed footing invert. In these cases the sampling may be performed at a closer interval, or perhaps even continuously. However, decreased sampling intervals may increase the duration of drilling and cost.

For thin-walled tube and stationary piston sampling in peat, silt or clay, there is no consistent sampling interval identified in practice. Since these methods can be used in conjunction with split spoon sampling, it is feasible to obtain undisturbed samples at any depth within a given strata. Therefore, selection of the sampling interval is generally based on engineering judgment given the soil conditions and the design issue.

Continuous sampling is used in Rhode Island most often for environmental and geotechnical characterization of surficial soils, particularly fill, above the water table. It is also used to identify stratagraphic changes such as the boundary between fill and native soils and the presence of varves in silts. In this method, a sampling tube is advanced into the soils at the bottom of a borehole. The sampling tube is recovered and the borehole is advanced through the sampled depth, and the process is repeated. Since the borehole is essentially open once the sample tube has been removed, a positive head of water or drilling fluid must be maintained in the casing to keep the hole open. Continuous samples can be obtained with both split spoon and thin-walled tubes, and the use may be used alternatively or in any sequence needed.

3.3.3 Test Pits

A test pit is an excavation that allows the upper zones of the soil profile to be exposed for characterization and sampling. The size and depth of the test pit depends on the information needed, equipment available, soil and water conditions, presence of boulders, and impacts on adjoining structures. A typical test pit is generally 3 feet by 8 feet minimum in the horizontal direction and approximately 10 feet below the ground surface. To allow safe inspection within the test pits, the slopes of the excavation must be cut back to stable slope angles or braced using structural elements. Bracing may also be considered in conditions where excavation may impact adjacent utilities or foundations. Test pits have been used to a limited extent in Rhode Island to characterize surficial soils (especially those containing fill, peat, and organics), as well as shallow bedrock. Test pits have also been used in evaluating the condition of existing building and bridge foundations.

3.3.4 Groundwater Observation

Groundwater observation is also an important aspect of geotechnical site investigations, and groundwater levels will impact both design considerations and excavation/construction activities. An understanding of water pressures and groundwater levels is necessary for evaluating effective stresses in soils, hydraulic forces on structures, and designing both temporary and permanent dewatering systems. Wells may be sited at proposed structure locations, detention/retention pond sites, or along drainage alignments. The intent is to allow short-term observations or evaluation of long-term and seasonal groundwater levels. Although water levels observed during drilling may be reported on boring logs, monitoring well observations are believed to be more representative of actual groundwater conditions. Well observations are used to assess construction dewatering needs and may be used in conjunction with soil sample testing, or pumping tests to assess and design dewatering systems. Depending on the accuracy that is needed, groundwater levels can be obtained from existing wells, boreholes during drilling, observation wells, or piezometers. Detailed information about the installation of observation wells and the measurement of groundwater levels can be found in ASTM D-4750, "Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well" and ASTM D-5092 "Design Installation of Ground Monitoring Wells in Aquifers."

Typically, observation wells consist of a small diameter slotted PVC pipe located within an open borehole, as shown in Figure 3.5. A probe is lowered into the observation well and used to locate the depth to the groundwater table. Generally, a boring is drilled to the desired well-depth using casing. The diameter of the borehole should be of sufficient diameter to allow for the well to be installed and clean sand to be backfilled around the slotted pipe. RIDOT specifies use of a minimum 10 foot long wellscreen; the wellscreen section is installed to span the water surface interval as observed during the drilling. The 10-foot length is intended to be sufficient to allow observations through anticipated dry and wet season conditions. The sand around the pipe acts as a filter to prevent clogging of the slotted pipe. A bentonite seal is then placed above the sand followed by a cement or grout mix to surface grade, and then the well is capped with a protective casing. The protective casing is grouted at the surface and acts as a guard against vandalism or unauthorized access.

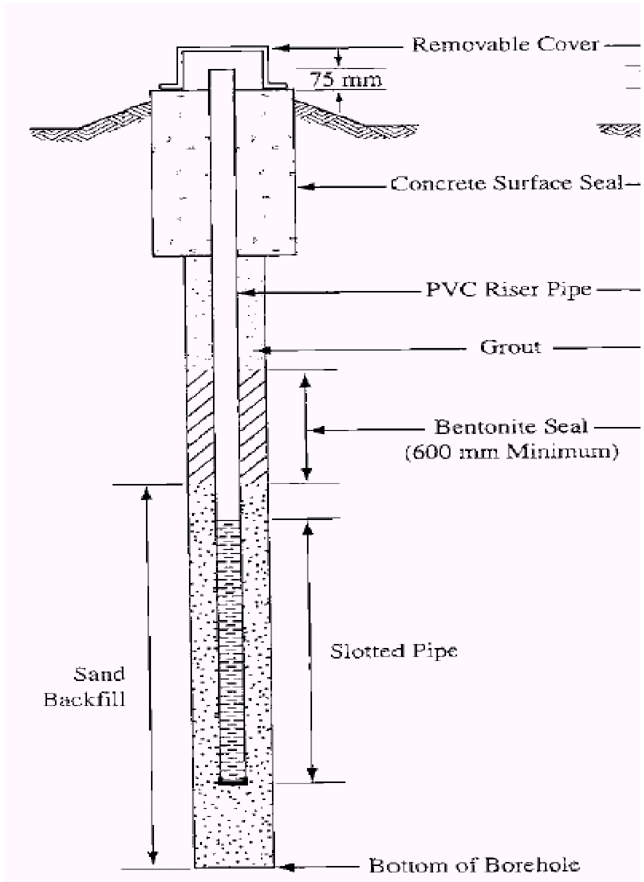


Figure 3.5. Typical observation well system (Arman et al. 1997).

3.3.5 Rock Classification

Many geotechnical designs require information about the depth to and quality of the underlying bedrock. The quality of the rock is generally assessed in terms of the properties of both the intact rock sample and the entire rock mass as a whole. The intact rock is classified based on a lithologic description of core samples collected in the field, as shown in Figure 3.6. This includes rock type, color, grain size and shape, texture, mineral composition, weathering, strength, and any other miscellaneous features observed. A typical description of a sample of limestone might read, "Limestone, light gray, very-fine grained, thin bedded, weathered, strong" (Arman et al. 1997). Detailed information on how to determine each of these characteristics can be found in the FHWA publication 97-021 1997 (Arman et al. 1997).

Figure 3.6. Example of rock samples collected during coring (http://sofia.usgs.gov/publications/posters/hydro_flkeys/geology.html).

Although the description of the intact rock is important, the overall behavior of the rock mass is governed by the degree of fractures, faults, joints, and seams in the rock mass. The pattern of the discontinuities observed in the rock cores are described according to type, attitude, spacing, tightness, planarity, regularity, continuity, and filling. The Rock Quality Designation (RQD) was developed as a means of quantifying the extent and nature of the discontinuities to assess the overall quality and condition of the rock mass. This is an on-the-spot characterization based upon core observations and simple measurements. RIDOT requires the RQD to be determined and entered on boring logs. The RQD is defined as the total length of core segments equal to or greater than 10 cm (4 in.) in length recovered from a borehole divided by the total length of core run. The RQD is expressed as a percent, which is a measurement of the quality of rock recovered from a borehole. This approach is illustrated in Figure 3.7 and Table 3.2.

There are other methods for evaluating the quality of the entire rock mass, including the Rock Mass Rating system (RMR), the NGI-Q system, and the Geological Strength Index (GSI). Of these, the RMR is often used in Rhode Island and will be described further. The Rock Mass Rating uses six parameters to evaluate rock quality: uniaxial compressive strength, rock quality designation (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions, and the orientation of the discontinuities (Mayne et al. 2001). Each parameter is assigned a numerical value (R_i) based on its quality, and the sum of the R-values equals the RMR. Ranges of the first five R-values and a summary of the RMR system are shown in Figure 3.8. The sixth parameter accounts for the orientation of the discontinuities to the proposed construction and can range from 0 to –60 depending on the situation. The RMR system is not performed routinely for RIDOT projects, but is sometimes of value where deep rock socketing of piles or drilled shafts might be anticipated.

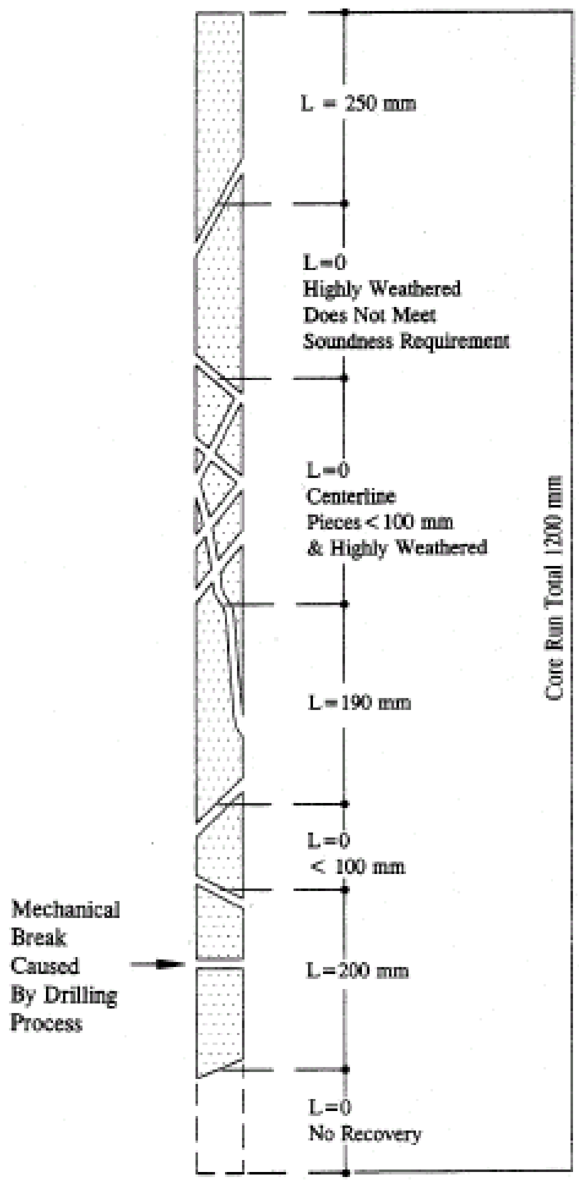
Table 3.2 Recommendations for determining the RQD of rock (Mayne et al. 2001b).

	Recommendations
Drilling Fractures	Only natural fractures should be considered for calculating RQD, as opposed to fractures due to drilling and handling.
Core Barrel Size and Type	Use Only NQ (2.98 in) size core or larger
Weathering	Rock assigned a weathering classification of moderately severe to very severe should not be used for RQD.
Core Recovery	Core recovery can vary from 100%, and RQD assumes 100% recovery therefore RQD should be determined on the basis of the total length of rock core recovered rather the total length of rock cored.
Centerline	The centerline measurement should be used because it is not dependent on the core diameter.
Assessment of Soundness	Pieces of core that are not “hard and sound” should not be counted for the RQD (i.e. where rock has been altered or weathered due to agents of surface weathering).

3.4 Site Investigations for the Design of Shallow and Deep Foundations

Both shallow and deep foundations distribute structural loads into the underlying soils without causing excessive settlements. This section presents a discussion of how to select the number and spacing of borings, appropriate boring depths, and sampling intervals to accurately characterize the soils beneath the foundation.

The aim of any geotechnical site investigation is to obtain sufficient and accurate information about the soil conditions necessary for the design of the foundation system. This information includes the types of soils beneath the footings, the locations of changes in strata, the depth to the groundwater table, and soil properties. Soils that may be of particular concern include organics (e.g. peat), uncontrolled fill, soft compressible clays and silts, and loose saturated sands. Soil properties are obtained either from laboratory tests on samples or from correlations with *in situ* test results like the Standard Penetration Test.



$$\text{RQD} = \frac{\sum \text{Length of Sound Core Pieces} > 100\text{ mm}}{\text{Total Core Run Length}}$$

$$\text{RQD} = \frac{250 + 190 + 200}{1200} \times 100\%$$

$$\text{RQD} = 53\%\ \text{(Fair)}$$

Rock Quality Description

RQD (Rock Quality Designation)	Description of Rock Quality
0 - 25%	Very Poor
25 - 50%	Poor
50 - 75%	Fair
75 - 90%	Good
90 - 100%	Excellent

Figure 3.7. Example of a how the RQD of a rock core is determined (Arman et al. 1997).

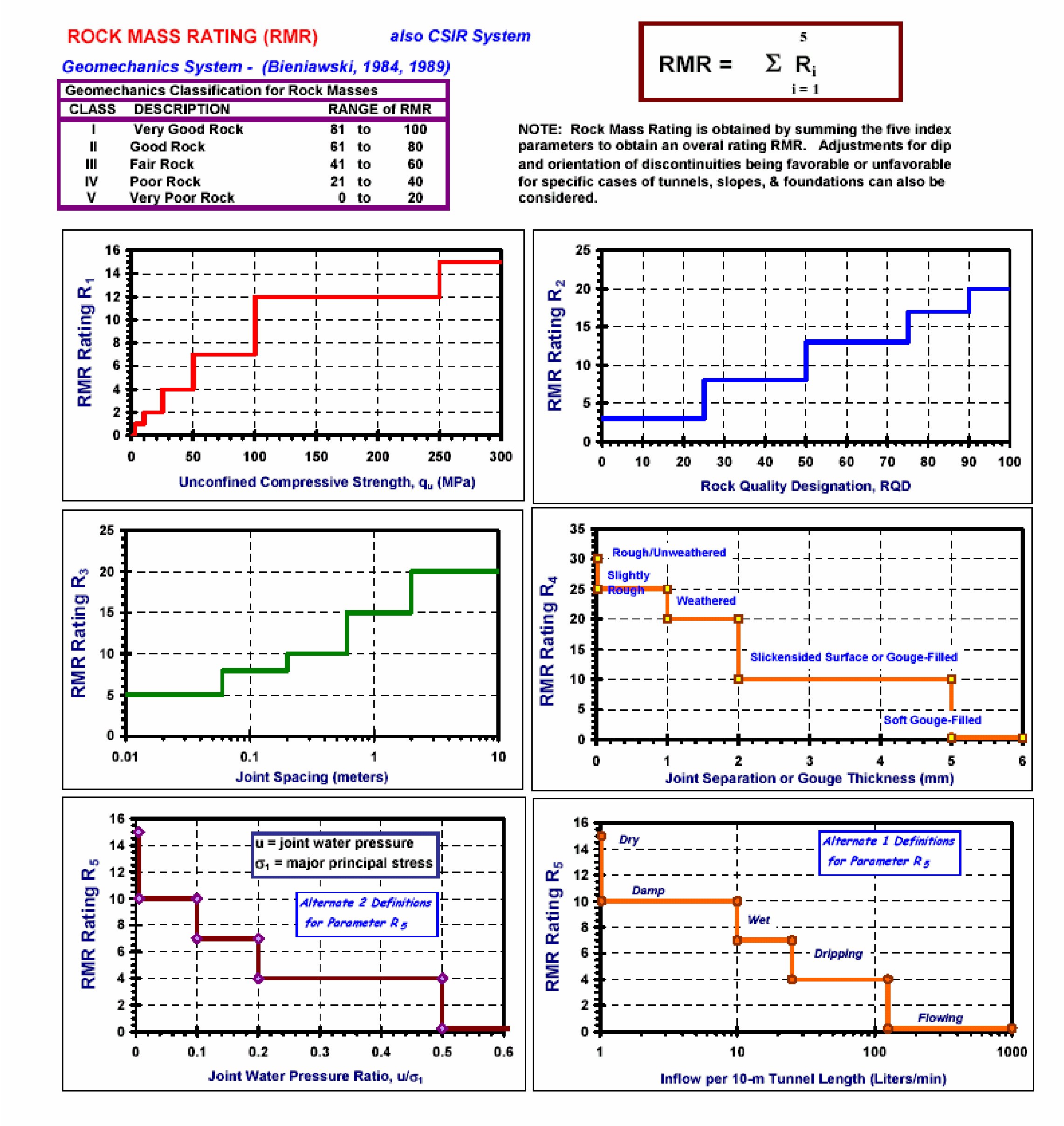


Figure 3.8. Rock Mass Rating System (Mayne et al. 2001).

3.4.1 Number and Spacing of Borings

Table 3.3 presents FHWA recommendations for the minimum number and spacing of borings for highway and bridge related projects. These include bridge foundations, walls, roadways, excavations, and culverts. These guidelines are simply minimum numbers and site investigation plans should always be done on a case-by-case basis. The number and spacing of borings should be selected to identify potential variations in the subsurface conditions, especially the depth to any bearing strata or

Table 3.3. Minimum Requirements for the Number and Spacing of Borings (from (FHWA HI 97-021 1997) and (Federal Highway Administration 1985)).

Area of Investigation	Number and Spacing of Borings
Bridge Foundations	A minimum of two borings should be performed for bridge piers or abutments over 100 ft (30 m) wide. A minimum of one boring should be performed for piers or abutments less than 100 ft (30 m) wide. Additional borings should be performed in areas of varying subsurface conditions. RIDOT typically performs a minimum of two borings for each abutment.
Retaining Walls	A minimum of one boring should be performed for each retaining wall. The distance between borings should be no greater than 200 ft (60 m) for retaining walls that are longer than 100 ft (30 m). In order to estimate lateral loads and the capacity of anchors, additional borings may be necessary inboard and outboard of the wall line to define conditions at the toe of the wall and in the active zone behind the wall.
Roadways	In general, the spacing of borings along a roadway alignment should not exceed 200 ft (60 m). The spacing and location of the borings should be selected based upon the geologic complexity and soil/rock boundaries within the project area, with the objective of defining the layering of distinct soil and rock units within the project limits.
Excavations and Embankments	A minimum of one boring should be performed for each cut slope. The spacing between borings for an excavation with a length greater than 200 ft (60 m) should generally be between 200 and 400 ft (60 and 120 m). At critical locations and at high cut areas, provide a minimum of three borings in the transverse direction to define the existing geological conditions for stability analysis. For an active slide, place at least one boring upslope of the sliding area.
Culverts	A minimum of one boring should be performed at each major culvert. Additional borings should be provided for long culverts or in areas of varying subsurface conditions.
Landslides	A minimum of two borings should be performed along a straight line perpendicular to the planned slope face to establish a geologic cross-section for analysis. The number of necessary cross-sections depends on the extent of the stability problem. For an active slide, place at least one boring above and below the sliding area.
Borrow Sources	Borings should be spaced every 100 to 200 ft (30 to 60 m).

bedrock. For example, if it is anticipated that the soil conditions will be highly variable in one direction across the site, a greater number of borings could be performed in that direction. Borings should be arranged so that they will provide the most appropriate information to the design engineers for the support of structures, excavation support, etc.

The number and spacing of borings should also reflect the size and type of structure. Borings can be conveniently placed at the location of pile groups or piers. For bridge projects in Rhode Island, for example, two borings per pier are sometimes used. However, for general purposes, a boring grid with a spacing ranging from 100 to 300 ft (30 to 90 m) is typical.

The necessary depth of borings will differ depending on whether shallow or deep foundations are used. However, it should be noted that the type of foundation is rarely known at the site investigation stage and borings should be deep enough to evaluate both foundation options.

3.4.2 Shallow Foundations

The bearing capacity and settlement of the footings govern the design of shallow foundations. In evaluating bearing capacity, it is necessary to obtain information regarding the location of the water table, and the density and the shear strength of the underlying soils. In evaluating settlement, it is also necessary to obtain the location of the water table and soil density, in addition to the compressibility behavior of the underlying soils. The characterization of silts, clays, and organics is particularly important because settlements of these soils under load can be large.

The appropriate depth of a boring and the sampling interval for shallow foundations can be related to bearing capacity and settlement analyses. The ultimate bearing capacity of a footing is determined using limit equilibrium methods, in which a specific failure surface is assumed and the soil strength along that failure surface is compared to the bearing pressure applied by the footing. The theoretical failure surface is shown in Figure 3.9. The depth of the assumed failure surface is approximately the width of the footing (B). Therefore, borings should penetrate at least a depth B beneath the footing to be able to evaluate the soil properties necessary for a bearing capacity analysis. In addition to split spoon samples, undisturbed samples should be taken in any

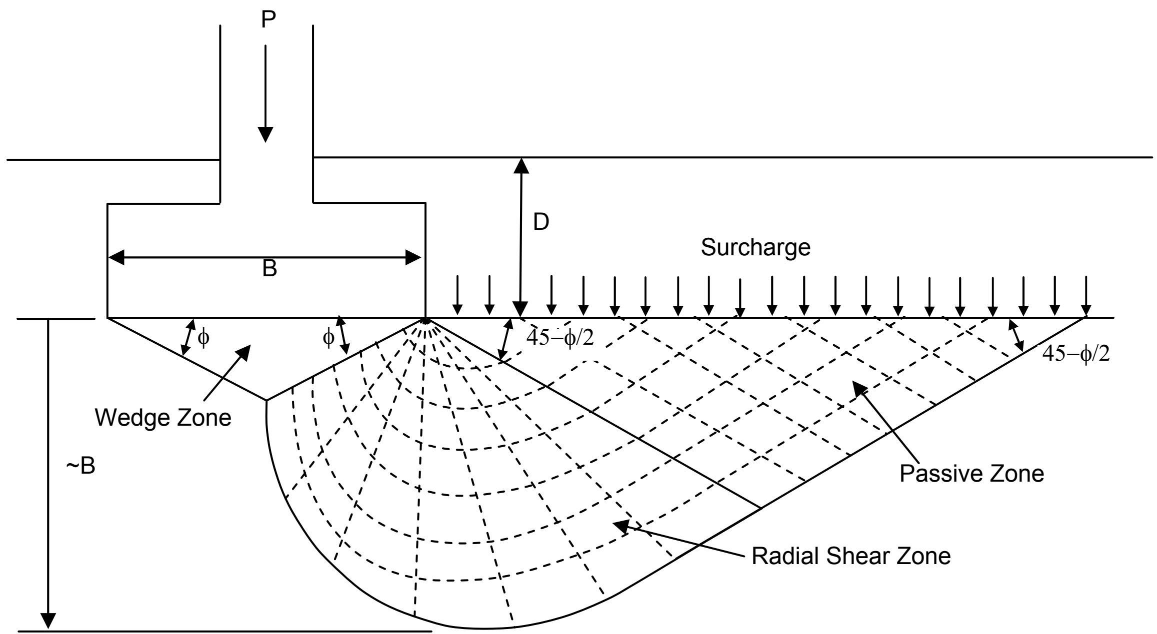


Figure 3.9 Assumed failure surface used to estimate the bearing capacity of a square footing. The depth of the failure surface is approximately equal to the width of the footing (after Das 1984).

cohesive deposits to characterize shear strength. A decreased sampling interval (i.e. less than 5 feet) or even continuous sampling may be considered in cases where B is relatively small.

The settlement of a shallow foundation is dependent on the compressibility of the soil and the magnitude of the bearing pressure. The increased stresses in the soil dissipate with depth and the distribution of applied stresses is often calculated using elastic theory. Figure 3.10 shows a typical distribution of the applied stresses with depth beneath a square and continuous footing. At a depth of approximately twice the footing width, the applied stress is 10% of the bearing pressure. Beneath this depth (2B), the increase in stress is so small that settlements are negligible. For continuous footings, settlements are negligible beneath a depth of four times the footing width (4B). Therefore, borings should penetrate a minimum depth of 2B for square footings and 4B for continuous footings in order to obtain soil properties for settlement analyses. In addition to split spoon samples, undisturbed samples should be taken in any cohesive deposits to characterize their compressibility.

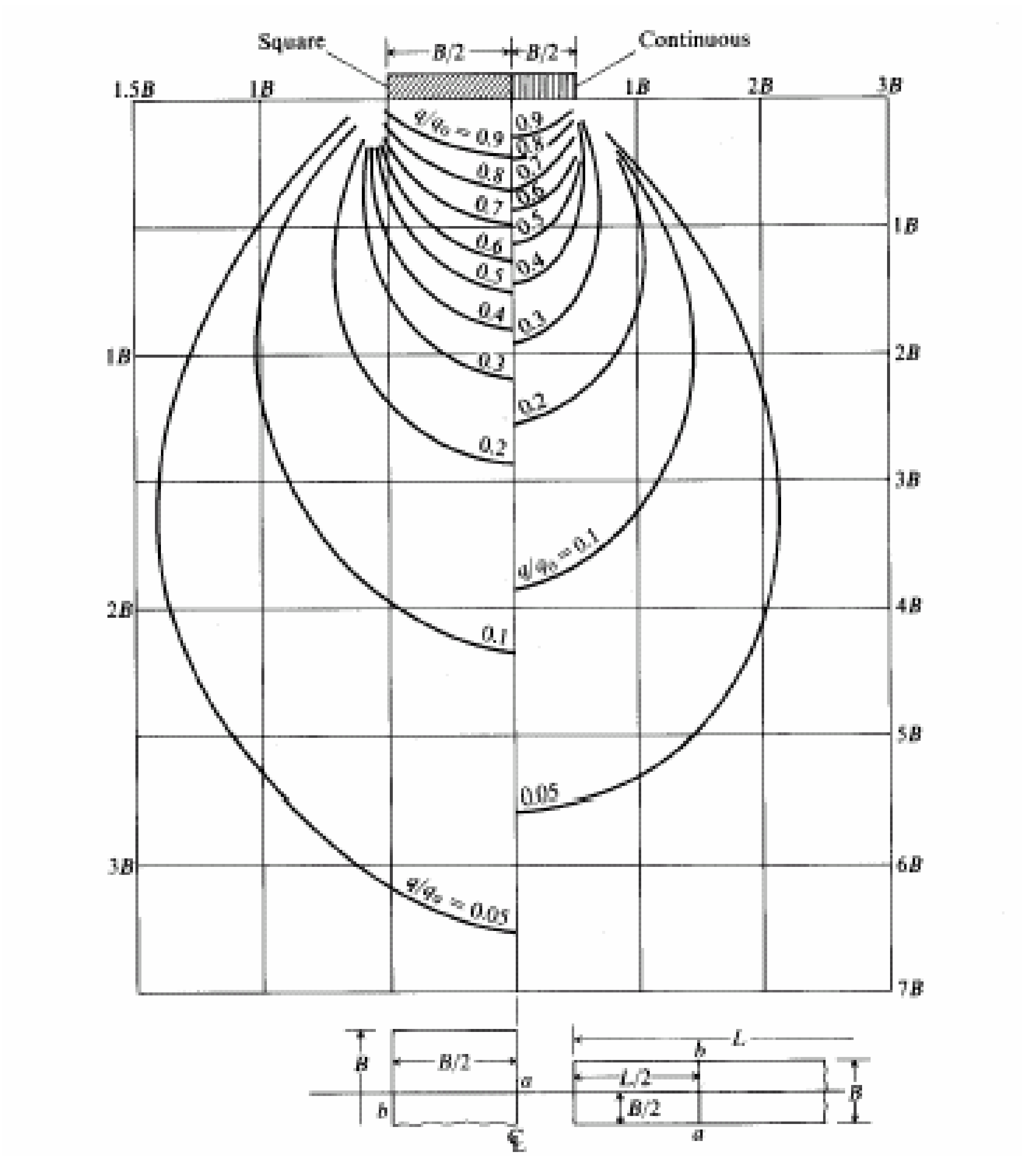


Figure 3.10. Typical stress distribution with depth for a square and continuous footing (Bowles 1988).

3.4.3 Deep Foundations

Deep foundations are used to transfer structural loads through weak or compressible soil layers to competent bearing soils or rock. Deep foundation systems (e.g. piles, drilled shafts) achieve their capacity through a combination of skin resistance and tip resistance. The information required for their design is similar to that needed for shallow foundations (i.e. types of soils, changes in strata, groundwater table, and soil properties), however borings for deep foundations are much deeper and in most cases extend into bedrock. It is important to identify soft compressible soils that may add load to the foundations as they settle (called downdrag) and obstructions like boulders or fill (urban fill with construction debris, old pipes, railroad ties, structural members, etc.) that can make installation of piles or drilled shafts difficult.

For deep foundations that are supported by soil, such as friction piles, stresses are distributed to the surrounding soil through skin friction along its length and end bearing resistance at its tip. The stress distribution at the tip of a pile is similar to that shown in Figure 3.10, and the Federal Highway Administration recommends that borings should extend below the anticipated embedment plus an additional 20 feet (6 m) or a minimum of 2 times the maximum pile group dimension (whichever is deeper) (Arman et al. 1997). A split-spoon sampling interval of 5 feet is generally considered appropriate for the design of deep foundations. In addition to split spoon samples, undisturbed samples should be taken in any cohesive deposits to characterize shear strength and compressibility.

For piles or shafts bearing on rock, the borings should extend a minimum of 10 feet (3 m) into the rock to ensure that the boring has not terminated on a boulder.

For drilled shafts extending into rock, the borings should extend below the anticipated shaft tip by a minimum of 10 feet (3m), or 3 times the shaft diameter for isolated drilled shafts.

3.5 RIDOT Recommended Procedures for Site Investigations

RIDOT has developed specific recommended drilling procedures for their projects, many of which are based on the general principles described above. These recommendations are listed below. Some of these items are required or specified in Section T of RIDOT Standard Specifications.

- Use cased, wash borings for all borings beneath proposed structures. Use minimum 3-in ID casing (NW size). Where fills or boulders are anticipated, it is recommended that the initial casing be a minimum 4-inch (HW size) diameter.

- Recirculation of the drilling fluid for wash borings is usually acceptable. Requiring non-recirculation of the drilling fluid may require consideration for containment, settlement and separate disposal of cuttings and used wash-water.
- Hollow-stem auger (HSA) borings may be used for relatively shallow highway and utility alignment explorations and explorations to define likely shallow bedrock along excavation alignments. HSA should not be used when there is significant penetration below the groundwater table or when recovery of undisturbed soil samples is needed.
- Continuous sampling or sampling intervals closer than the standard 5-foot maximum should be used to define the interfaces between fills and natural soils (not always evident) and the extent of organic or other compressible strata.
- Shelby, hydraulic piston (e.g. Guss, Osterberg), or fixed-piston tube samplers should be used to recover undisturbed samples of silt or organic soils for laboratory determination of engineering properties. They should be used in conjunction with Standard Penetration Test (SPT) immediately above and below the interval of undisturbed sample attempts.
- Where rock coring is not proposed or practical due to extreme depths, the drilling schedule may require ending the boring at a particular depth or continuation of the drilling until "refusal" to further penetration of driven casing and the SPT sampler. Alternatively the schedule might require continuation to such depth where till is encountered. "Refusal" is a non-specific term, which generally refers to that point in the borehole penetration at which no deeper penetration can be made by standard techniques (driving casing under 300-lb hammer blows). When applied to the SPT, "refusal" may be variably defined as material requiring more than 100 blows to penetrate 12 inches, 50 or more blows to penetrate 6 inches, etc. This should be clearly defined on proposed drilling plan notes.
- When a boring reaches the proposed termination depth, the recovered sample and blow counts of the final SPT should be reviewed. If there is little recovery or the soils consist of loose to very loose materials ($N \leq 10$ blows per foot), continue borings and SPT to depths such that a minimum of 10 to 15 feet of soil exhibits SPT N-values ≥ 20 blows per foot.

- Simply experiencing high blow counts is no guarantee that bedrock or dense till has been encountered. Local soil strata other than till often contain boulder or cobble sized materials that may impede or stop standard drilling and sampling methods. It is recommended that in such instances, further penetration should be attempted using a roller bit (a drill bit with hardened rollers at the tip) under down-pressure (the driller will likely do this as a matter of course). If there is no further penetration, the boring should be continued using rock coring equipment and techniques. The initial intent is to determine whether the obstruction is caused by a cobble or boulder, very dense and indurated soils as may be characteristics of the local till, or if bedrock surface has been reached. RIDOT policy is to require a minimum 10-foot length core of the encountered material. If the coring breaks through the obstructing material or boulder, the boring should continue to be advanced using standard drilling methods. The 10 foot minimum core length requirement assumes that boulders larger than 10 feet in diameter are rare. Judgment must be used to evaluate the type of rock recovered and the depth at which the material was encountered, and compare these observations with local experience or other borings done in the immediate area.
- There are cases where the deep foundation design will include a length of pile or drilled shaft to be "socketed" in rock. The length of the needed rock socket will be a function of the design shaft diameter, the thickness of unsuitable overlying soils, or the character (rock type and strength properties) of the penetrated rock itself. In these instances, scheduled rock coring penetration may be greater than 10 feet.

3.6 Typical Drilling Production Rates

Production rates will vary depending upon the type of drilling (i.e. hollow stem auger vs. rotary wash), scheduled depths of borings, SPT and soil sampling schedule (standard 5 foot interval or continuous), scheduled depth of rock cores, well installations, and other factors. Production rates for water borings will generally be less than drilling on land beyond the time for mobilization on site. Debris (concrete and wood in fills), boulders, and broken or very hard rock (coring time increases), equipment breakdowns, and down-hole tool losses will also impact drilling progress. Traffic control equipment set up and break down will reduce time actually spent drilling during the day. Additionally, it is RIDOT policy to limit work-time and occupancy on major roads during high-volume traffic periods.

Given these factors, hollow stem auger borings (including split spoon SPT sampling) have the highest production rates. If drilling locations are relatively close, accessible to truck-mounted rigs, and the depths are limited to approximately 20 feet, approximately 6 to 8 borings may be produced in the course of an 8-hour work day.

The production of rotary wash borings with casing may be better assessed based upon footage and required depths for each boring. If borings are approximately 20 feet deep with SPT sampling at 5-foot intervals and no rock coring, then 3 to 5 borings can typically be completed in the course of one day. However, as the drilling depth increases to 50 to 70 feet, approximately 1 to 1.5 borings can be accomplished per day. Production rates decrease with depth at each location due to handling (raising and lowering) of increasing tool lengths associated with standard interval sampling. Rock coring typically takes ½- to ¾-hours for each 5-foot length of core, including lowering the core barrel, drilling rock, and recovering the cored sample. It should be noted that these times and rates are approximate, and actual production will vary due to particular site conditions and subsurface soils.

3.7 Typical Costs for RIDOT Site Investigations

The cost of geotechnical site investigations for individual projects can range from on the order of $10,000 to more than $100,000. The design of large projects such as the Washington Bridge Rehabilitation, relocation of Rte I-195 in Providence, and relocation of Rte 403 through North Kingstown have required multiple and typically deep borings. State regulations require that site investigations with anticipated or estimated budgets (drilling only) greater than $100,000 require that the proposed drilling be advertised for bid.

Drilling costs for typical bridge construction or rehabilitation projects range from $10,000 to $40,000. These are costs of the drilling only, i.e., the drilling contractor’s equipment, materials, and labor expended to perform the drilling. Consultant costs associated with preparation of bid documents, plans, drilling schedule, monitoring the drilling, and survey of "as drilled" borehole locations and elevations are not included.

Items typically considered in a drilling program are shown in Table 3.4. These are taken directly from the RIDOT "B-pages." and illustrate the range of tasks commonly performed for a DOT sponsored site investigation.

Table 3.4 Bid items for RIDOT site investigations.

Bid Item	Unit Measurement
3 inch min. diameter soil borings using a truck rig or skid rig (ON LAND)	Linear Feet
3 inch min. diameter soil borings using a truck rig or skid rig (ON WATER)	Linear Feet
3 inch min. diameter soil borings using drilling mud	Linear Feet
4 inch min. diameter soil borings using a truck rig or skid rig	Linear Feet
AX (1.185 in diameter) rock coring	Linear Feet
NX (2.154 in diameter) rock coring (ON LAND)	Linear Feet
NX rock coring (ON WATER)	Linear Feet
Hollow stem auger borings	Linear Feet
Test Pits	Each
Bar Soundings	Linear Feet
Pipe Probings	Linear Feet
One-inch Retractable Piston Sampler Boring	Linear Feet
Additional Split-Spoon Sample (ON LAND)	Each
Additional Split-Spoon Sample (ON WATER)	Each
Thin-Wall Sample	Each
Stationary Piston Thin-Wall Sample	Each
Packing, Freezing, and Shipping One-Inch Retractable Piston Samples	Each Series
Field Vane Shear Test	Each
Groundwater Observation Wells	Linear Feet
Inclinometer Casing Installed	Linear Feet
Coring Concrete Slabs and Bituminous/Concrete Pavement	Each
Mobilization and Demobilization of Crew and Drill Rig	Each
Traffic Control Equipment	Location
Traffic Person(s)/Flag Person(s)	Hourly
Railroad Permit	Lump Sum
Railroad Flag Person(s)	Daily

4. Standard Penetration Test

4.1 Introduction

The Standard Penetration Test (SPT) is the most widely used in situ test and sampling device in the United States. The sampler is typically an 18 to 30 inch long split-barrel tube (commonly called a "split-spoon") with an inside diameter of 1.5 to 1.375 inches, as shown in Figure 4.1. The split-spoon is attached to the end of type A or AW drill rod and driven into the ground with a 140 lb hammer. The SPT yields two important pieces of information for geotechnical site investigations: a disturbed soil sample and the resistance to penetration of the split-spoon. The sample is used to classify soils at the site, estimate the position of the groundwater table, and identify changes in soil strata. It is essential that samples are obtained to verify, or "ground-truth," other geotechnical or geophysical measurements. The penetration resistance is measured by the number of blows of the hammer it takes to drive the split-spoon a specific distance, usually 12 inches, into the soil. The number of blows per foot of penetration is referred to as the N-value of the SPT. This value is an indication of soil consistency or density and is used extensively for the design of foundations, assessment of soil properties, and the evaluation of liquefaction potential at a site. Estimates of soil strength and compressibility, bearing capacity, and settlement can all be made based on N-values. However, one must be careful about using blow counts directly because N-values can be influenced significantly by the type of hammer used, length of the drill rods, overburden stress of the soil, operator technique, and a variety of other factors.

This chapter summarizes the state-of-the-practice on the use and interpretation of data obtained from the Standard Penetration Test. First, a list of selected publications on this topic will be provided for reference. A description of the SPT equipment and procedures will be presented along with the factors that influence the measured N-values. Corrections that can be applied to the measured N-values will also be discussed along with the advantages and disadvantages of the SPT. Finally, selected empirically based correlations will be presented that relate blow counts to specific engineering properties.

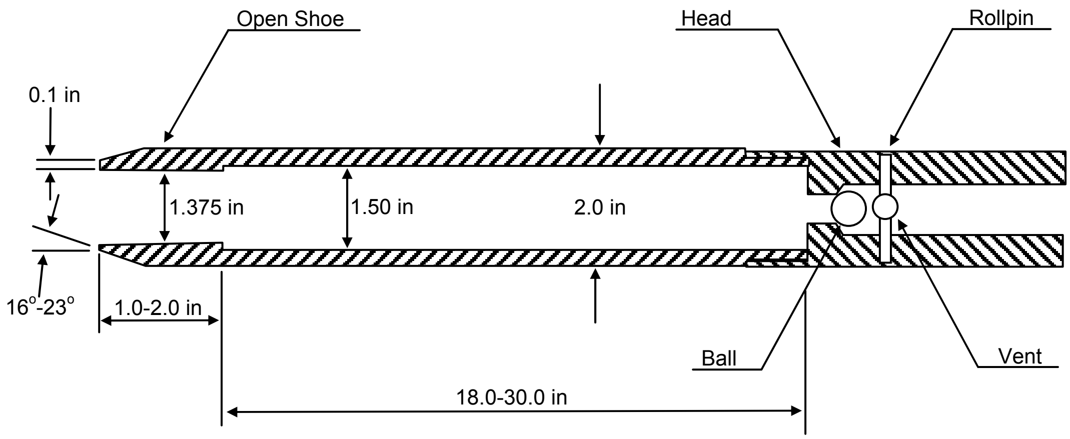


Figure 4.1. Split barrel sampler with dimensions (after ASTM D 1586).

4.2 Selected Publications on the Standard Penetration Test

Although there are many references sited in this chapter, four publications were used heavily in its development. These publications are listed below and can be used for further reference.

- Arman, A., Samtani, N., Castelli, R., and Munfakh, G. (1997). Geotechnical and Foundation Engineering Module 1 – Subsurface Investigations, FHWA-HI-97-021, 305 pp.
- Carter M. and Bentley, S. P. (1991). Correlations of Soil Properties. London, Pentech Press Publishers, London, 130 pp.
- McGregor, J. A. and Duncan, J. M. (1998). Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice. Center for Geotechnical Practice and Research, Virginia Tech.
- Naval Facilities Engineering Command (1982). Soil Mechanics Design Manual 7.1. DM-7.1.

4.3 General Equipment and Procedures

The recommended procedure for performing the SPT is defined by the American Society for Testing and Materials specification D1586 entitled "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils." The test first involves the opening of a borehole using an auger (solid stem or hollow stem) or a roller bit (drill bit with hardened rollers at the tip) to a desired depth. It is important to observe the action of the drill during penetration, as it can be an indication of the type of materials that are encountered or a change in stratigraphy. For example, shaking of the drill rods may be an indication of gravel or cobbles. When the bottom of the hole is cleaned, the sampler is then lowered to the bottom of the hole at the end of type A or AW drill rod. A list of drill rod dimensions is shown in Appendix B. A 140 lb hammer and anvil is attached to the top of the drill rods and the split-spoon sampler is driven into the ground by successive dropping of the hammer 30 inches onto the anvil.

The test is completed when the split spoon penetrates 18 to 24 in. or until 100 blows have been applied. The number of blows is recorded for each 6 in. interval. The upper 6 inches of material is considered to be disturbed and is generally neglected. The number of blows for 12 inches of penetration from 6 to 18 in. intervals, and is reported as the "N-value" or standard penetration resistance. The sampler is then retrieved, and the soil sample is removed and classified. The drill is then advanced to the next depth interval (usually 2.5 to 10 feet) and the process continues until the specified drill depth is achieved. The acquired N-values are generally presented on a boring log along with the soil classification.

Three different types of hammers are used for the Standard Penetration Test in the United States. Donut and safety hammers, shown in Figure 4.2, are the most commonly used. The safety hammer delivers approximately 60% of the maximum free-fall energy (140 lb weight dropped a distance of 30 in) to the drill stem. The donut hammer delivers approximately 45% of the maximum free-fall energy to the drill stem. The automatic hammer, shown in Figure 4.3, delivers 95% to 100% of the maximum free-fall energy to the drill stem. Knowledge of the energy of the hammer is extremely important because the measured blow count is directly related to the energy transferred from the hammer to the drill rods, and corrections can be applied to standardize the reported N-values. The most common method of raising and lowering the donut and safety hammer is by a rope that is wrapped around a spinning drum, called a cathead. Proper methods of wrapping the rope around the cathead are shown in Figure 4.4. The automatic hammer is raised and lowered by a mechanism at a preset blow count frequency.

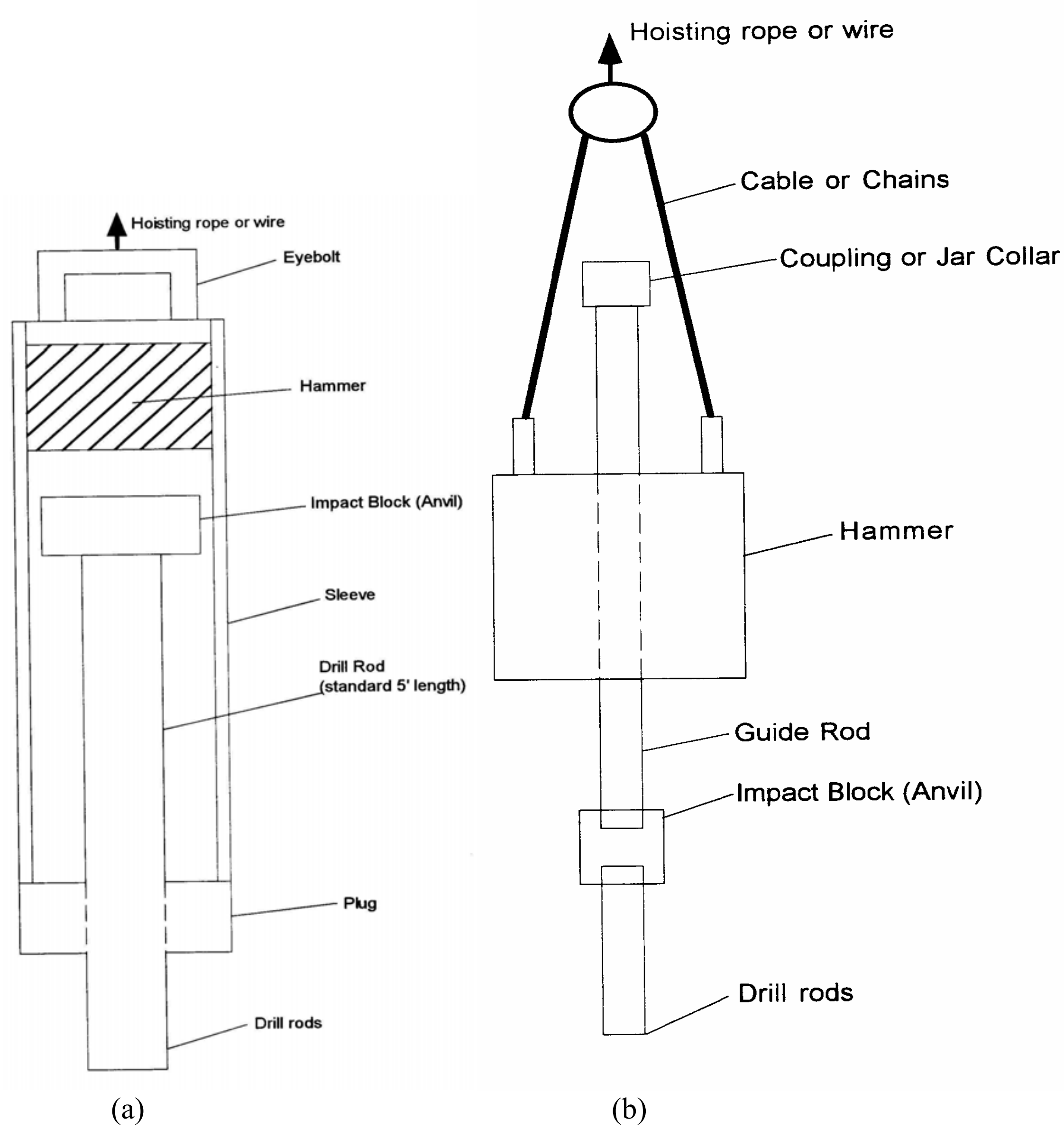


Figure 4.2. a.) Safety hammer and b.) Donut hammer (McGregor and Duncan 1998).

4.3.1 Additional Recommended Equipment and Procedures

ASTM D1586 allows for significant variations in drilling techniques, equipment dimensions, and hammers. Even the split spoon can vary in length, diameter, and angle of the nose cone. This is why it is essential that all equipment and procedures be carefully documented in the field so that the appropriate corrections or interpretations of the N-values can be made. Though the methods defined by ASTM D1586 are considered acceptable in engineering practice, some specific equipment and procedures have been proven to give more accurate N-values. These methods are described below.

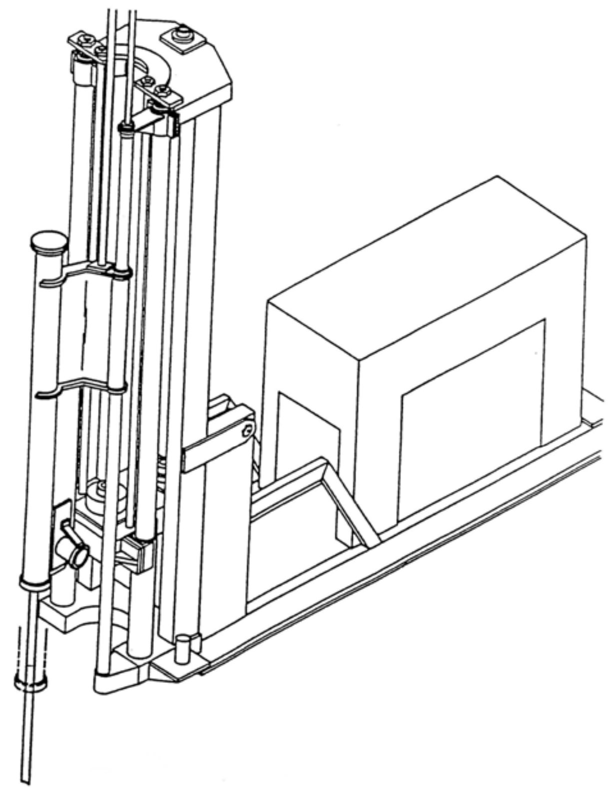


Figure 4.3. Automatic hammer (McGregor and Duncan 1998).

Skempton (1986) made the following recommendations to obtain accurate N-values:

- The wash boring method or rotary drilling with a tricone bit should be used to minimize soil disturbance.
- Water or drilling mud in the borehole should be used to minimize the reduction in vertical effective stress within the soil at the sampling location. Water and drilling mud must be maintained at or above the groundwater table.
- The bottom of the boring should be between 2.5 and 6 in. in diameter, although a maximum diameter of 4 in. is preferred.
- Casing should not extend below the bottom of the boring before the SPT is performed.

- The measured N-value should be taken from the penetration between 6 and 18 inches. The first 6 in. below the bottom of the boring is considered to be disturbed material.

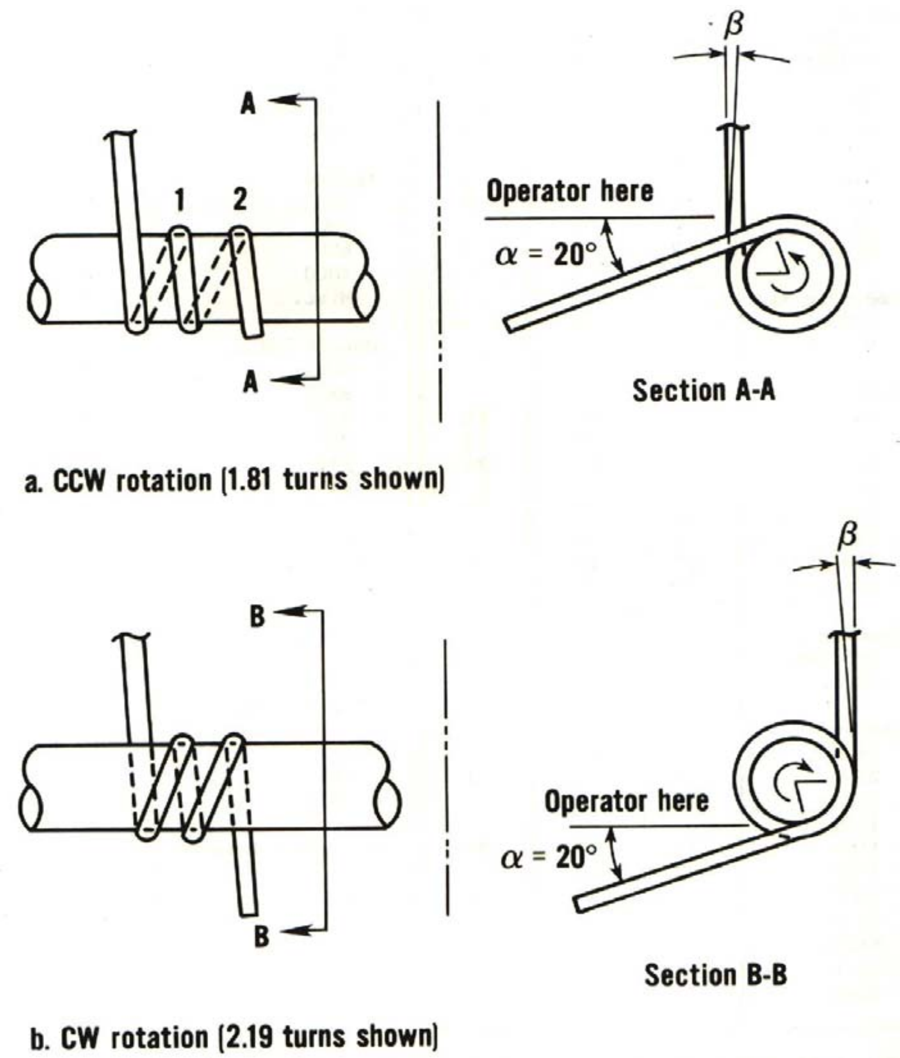


Figure 4.4. Rotation of rope around a cathead (ASTM D 1586).

Seed et al. (1984) made recommendations for standardizing the SPT procedure for the assessment of liquefaction potential. These recommendations include the following:

- SPT N-values should be corrected for 60% of the maximum free-fall energy (140 lb weight dropped a distance of 30 in) to the drill stem. This corresponds to the energy delivered by the safety hammer.
- The diameter of the boring should be 4 – 5 in.

- Drilling mud should be used to prevent liquefaction of loose sands (sometimes called "running sands") below the water table.
- A drilling bit should be used that produces an upward deflection of the drilling mud.
- The split spoon sampler should have a uniform inside diameter of 1 3/8".
- A or AW rods should be used for borings less than 50 ft (see Appendix B for drill rod sizes). Use N or NW rods for depths greater than 50 ft.
- A blow count rate of 30 to 40 blows/minute should be used.

Dr. William Kovacs at the University of Rhode Island has performed considerable research on the accuracy and precision of the SPT (Kovacs (1977); Kovacs (1980); Kovacs and Salomone (1982); Kovacs (1994)), with particular focus on the energy transfer of the different hammers. He recommends the following:

- Use of the automatic hammer that delivers 95% to 100% of the maximum free-fall energy to the drill stem.
- Automatic hammers should be adjusted to deliver 60% of the maximum energy to correspond to historical N-values obtained with safety hammers.
- If a cathead and rope device is used, two turns around the cathead are used. Also, the following information should be recorded on boring logs to aid in the interpretation of results: number of turns of the rope around the cathead, the direction of rotation of the cathead, and the approximate angles of rope with respect to the horizontal and vertical planes.

4.4 Factors Affecting Measured N-values

There are many factors that can affect the measured value of penetration resistance from the SPT. These factors can either increase or decrease the N-values, and can significantly affect the assessment of soil properties at a site. An understanding of these factors can be especially useful to the engineer in the field where observations can be made and corrective actions to the drilling activities implemented.

Fletcher (1985) first identified significant causes of error in the SPT and the factors that can affect measured N-values. The following list of factors is taken directly from Fletcher's text:

- Inadequate cleaning of the borehole;
- Failure to maintain sufficient hydrostatic head in the boring;
- Variations from an exact 30 in. drop of the drive weight;
- Use of drill rods heavier than 1 in. extra heavy pipe or A rods;
- Extreme length of drill rods (over 175 ft);
- Interference with free fall of the drive weight from any cause;
- Use of 140 lb weight without hardwood cushion, block or guide rod;
- Use of sliding weight that can strike the drive cap eccentrically;
- Use of deformed tip on sample spoon;
- Excessive driving of sample spoon before the blow count;
- Failure to seat sample spoon on undisturbed material;
- Driving of sample spoon above bottom of casing;
- Carelessness in counting the blows and measuring penetration.

Additional factors that affect measured N-values can be found in NAVFAC DM 7.1 (1982), Decourt (1990), and Kulhawy and Trautmann (1990). Table 4.1 is taken directly from the NavyDesign Manual and illustrates the effect of improper testing procedures on measured N-values. Decourt (1990) published a state-of-the-art report on the SPT at the 12th International Conference on Soil Mechanics and Foundation Engineering. Factors that affect the penetration resistance that were identified in this report are shown in Table 4.2. The work performed by Kulhawy and Trautmann (1990) is summarized in Table 4.3

Table 4.1. Factors that affect measured N-values

(NAVFAC DM 7.1 1982).

Factors	Comments
Inadequate cleaning of the borehole	SPT is only partially made in original soil. Sludge may be trapped in the sampler and compressed as the sampler is driven, increasing the blow count. (This may also prevent sample recovery.)
Not seating the sampler spoon on undisturbed material	Incorrect N-values obtained.
Driving of the sample spoon above the bottom of the casing	N-values are increased in sands and reduced in cohesive soils.
Failure to maintain sufficient hydrostatic head in boring	The water table in the borehole must be at least equal to the piezometric level in the sand, otherwise the sand at the bottom of the borehole may be transformed into a loose state thereby decreasing the blow counts
Attitude of operators	Blow counts for the same soil using the same rig can vary, depending on who is operating the rig, and perhaps the mood of operator and time of drilling.
Overdrive sampler	Higher blow counts usually result from an overdriven sampler.
Sampler plugged by gravel	Higher blow counts result when gravel plugs the sampler, resistance of loose sand could be highly overestimated.
Plugged casing	High N-values may be recorded for loose sand when sampling below groundwater table. Hydrostatic pressure can cause sand to rise within the casing.

Table 4.1 (continued).

Factors	Comments
Overwashing ahead of casing	Low blow count may result for dense sand since overwashing loosens sand.
Drilling method	Drilling technique (e.g., cased holes vs. mud stabilized holes) may result in different N-values for the same soil.
Free fall of the drive weight is not attained	Using more than 1-1/2 turns of rope around the drum and or using wire cable will restrict the fall of the drive weight.
Not using correct weight	Driller frequently supplies drive hammers with weights varying from the standard by as much as 10 lbs.
Weight does not strike the drive cap concentrically	Impact energy is reduced, increasing N-values.
Not using a guide rod	Incorrect N-value obtained.
Not using a good tip on the sampling spoon	If the tip is damaged and reduces the opening or increases the end area the N-value can be increased.
Use of drill rods heavier than standard	With heavier rods more energy is absorbed by the rods causing an increase in the blow count.
Not recording blow counts and penetration accurately	Incorrect N-value obtained.
Incorrect drilling procedures	The SPT was originally developed from wash boring techniques. Drilling procedures which seriously disturb the soil will affect the N-value, e.g. drilling with cable tool equipment.
Using drill holes that are too large	Holes greater than 10 cm (4 in) in diameter are not recommended. Use of larger diameters may result in decreases in the blow count.

Table 4.1 (continued).

Inadequate supervision	Frequently a sampler will be impeded by gravel or cobbles causing a sudden increase in blow count; this is not recognized by an inexperienced observer (Accurate recording of drilling, sampling, and depth is always required).
Improper logging of soils	Not describing the sample correctly.
Using too large a pump	Too high a pump capacity will loosen the soil at the base of the hole causing a decrease in blow count.

Table 4.2. Factors that affect N-values (Decourt 1990).

Factors that Affect Measured N-Values	Effect on N-values
Different methods of lifting and releasing the hammer	Increase or decrease
Variations from the exact 30 inch drop	Increase or decrease
Failure of driller to completely release the tension of the rope	Increase
Use of wire line rather than manila rope	Increase
Insufficient lubrication of the pulley	Increase
Attitude of operators	Increase or decrease
Not using correct weight	Increase or decrease
Not striking the anvil concentrically	Increase
Not using a guide rod	Increase
Not recording blow counts and penetration resistance accurately	Increase or decrease
Inadequate cleaning of disturbed material	Decrease
Failure to maintain sufficient hydraulic head	Decrease
Larger size borehole	Decrease
Using a pump of too high capacity	Increase

Table 4.2 (continued).

Drilling mud instead of casing (in sands)	Increase
Weight of drill rods	Decrease
Deformed sampler	Increase
Driving sampler spoon above the bottom of the casing (in sands)	Increase
Increase in anvil weight	Decrease
Sampler plugged by gravel	Increase
Large I.D. for liners but no liners	Decrease

Table 4.3. Relative significance of various factors on measured N-values (from Kulhawy and Trautmann 1996).

Factors That Affect N-values	Relative Significance
Size and weight of drill rods	Penetration resistance is not affected
Non standard sampler	Moderate effect on test results
Deformed or damaged sampler	Moderate effect on test results
Rod Length	Minor effect on test results
Drill Rig Type	Minor effect on test results
Hammer Weight	Minor effect on test results
Rod diameter/weight	Minor effect on test results
Error in Counting Blows	Minor effect on test results
Deformed drill rods	Minor effect on test results
Hammer Type	Moderate to significant effect on test results
Borehole Cleaning	Moderate to significant effect on test results
Insufficient Hydrostatic Head	Moderate to significant effect on test results
Seating of Sampler	Moderate to significant effect on test results
Hammer Drop Method	Moderate to significant effect on test results
Anvil Size	Moderate to significant effect on test results
Hammer Drop System	Significant effect on test results

4.5 Corrections to Measured N-values

As discussed in the previous section, numerous factors can affect the measured N-value from a Standard Penetration Test. Corrections can be applied to the raw data to remove certain influences and normalize the N-values so that data obtained using different equipment or at different depths can be directly compared. The corrections that are applied to the measured blow count to obtain the corrected, $N_{1\,(60)}$ blow count is shown in Equation 4.1 (Youd and Idriss 1997). $N_{1\,(60)}$ values are more commonly used in empirical correlations to obtain soil properties and in the assessment of liquefaction potential.

$$N_{1\,(60)} = N_m\, C_N C_E\, C_B C_R\, C_S\, C_A\, C_{BF}\, C_C \qquad (4.1)$$

where $N_{1\,(60)}$ = measured blow count corrected to 60% of the theoretical free-fall hammer energy, 1 tsf effective overburden pressure, and other factors;

N_m = measured blow count in the field;

C_N = overburden correction factor;

C_E = energy correction factor;

C_B = borehole diameter correction factor;

C_R = rod length correction factor;

C_S = sampling method (liner) correction factor;

C_A = anvil correction

C_{BF} = blow count frequency correction factor; and

C_C = hammer cushion correction factor

The most widely applied corrections are for overburden stress (C_N) and transmitted energy of the hammer (C_E). SPT's performed at depth in a uniform soil deposit will yield higher N-values than shallow tests due to the increased confinement of the overlying soils (vertical effective stresses increase with depth). Therefore, the overburden stress correction normalizes the measured N-value in the field at any depth to a reference stress of 1 tsf (100 kPa).

The purpose of the energy correction is to account for tests performed using different types of hammers (e.g. safety, donut, automatic). The safety hammer delivers approximately 60% of the maximum free-fall energy (140 lb weight dropped a distance of 30 inches) to the drill stem. The donut hammer delivers 45% of the maximum free-fall energy, and the automatic hammer delivers 95% to 100% of the maximum free-fall energy to the drill stem. The measured N-value is normalized to the energy transmitted by the safety hammer (60%), and the correction factor is defined as:

$$C_E = \frac{ER}{60} \tag{4.2}$$

where ER = energy ratio (typically 60 for safety hammer, 45 for donut hammer, 100 for automatic hammer).

Recommended correction factors are shown in Table 4.4. **It should be noted that many of these factors are not routinely applied in geotechnical site investigations in Rhode Island. A survey of geotechnical engineering firms in the area found that corrections are applied mostly for the analysis of liquefaction potential. In these cases N-values are corrected for overburden stress and hammer energy.**

Table 4.4. Recommended correction factors for the SPT
(Youd and Idriss 1997; Skempton 1986).

Factor	Equipment Variable	Term	Correction
Overburden stress	---	C_N	$\left(\frac{P_a}{\sigma'_v}\right)^{0.5}$ for $C_N \leq 1.7$ for $C_N > 1.7$, use $C_N = 1.7$
Energy ratio	Donut hammer	C_E	0.5-1.0
Energy ratio	Safety hammer	C_E	0.7-1.2
Energy ratio	Automatic hammer	C_E	0.8-1.3
Borehole diameter	2.5 – 4.5 in	C_B	1.0
Borehole diameter	6 in	C_B	1.05
Borehole diameter	8 in	C_B	1.15
Rod length	< 9.8 ft	C_R	0.75
Rod length	9.8 – 13.1 ft	C_R	0.8
Rod length	13.1 – 19.7 ft	C_R	0.85
Rod length	19.7 – 32.8 ft	C_R	0.95
Rod length	32.8 – 98.4 ft	C_R	1.0
Sampling method	with liners	C_S	1.0
Sampling method	without liners	C_S	1.3
Anvil	Donut hammer	C_A	0.7-0.85
Anvil	Safety hammer	C_A	0.9-1.0
Blow count frequency (saturated sands only)	$N_{1(60)} < 20$ 10-20 bpm	C_{BF}	0.95
Blow count frequency (saturated sands only)	$N_{1(60)} > 20$ 10-20 bpm	C_{BF}	1.05
Hammer cushion	none	C_C	1.0
Hammer cushion	New, hard wood	C_C	0.95
Hammer cushion	Used, hard wood	C_C	0.9

A range of values for these correction factors has been published in the literature. In particular, the correction factors C_N, C_R, C_B, and C_S, are described in more detail below.

4.5.1 Overburden Correction Factor (C_N)

The overburden correction factor recommended by Youd and Idriss (1997) in Table 4.4 was originally recommended in a slightly different form by Liao and Whitman (1986). This and other published overburden correction factors are listed in Table 4.5.

Table 4.5. SPT Overburden Correction factors
(as reported by Carter and Bentley 1991).

<table>
<tr><th>Reference</th><th colspan="2">Correction Factor (C_N)</th><th>Units of Overburden Stress (σ'_v)</th></tr>
<tr><td>Gibbs and Holtz (1959)</td><td colspan="2">$C_N = \frac{50}{10+\sigma'_v}$</td><td>psi</td></tr>
<tr><td>Bazaraa (1967)</td><td colspan="2">$C_N = \frac{4}{1+2\sigma'_v}$ when $\sigma'_v \leq 1.5$
$C_N = \frac{4}{3.25+.5\sigma'_v}$ when $\sigma'_v > 1.5$</td><td>ksf</td></tr>
<tr><td>Peck, Hanson, and Thornburn (1974)</td><td colspan="2">$C_N = .77\log_{10}\frac{20}{\sigma'_v}$</td><td>kg/cm^2, tsf</td></tr>
<tr><td>Seed (1976)</td><td colspan="2">$C_N = 1-1.25\log_{10}\sigma'_v$</td><td>kg/cm^2, tsf</td></tr>
<tr><td>Tokimatsu and Yoshimi (1983)</td><td colspan="2">$C_N = \frac{50}{10+\sigma'_v}$</td><td>kg/cm^2, tsf</td></tr>
<tr><td>Liao and Whitman (1986)</td><td colspan="2">$C_N = \sqrt{\frac{1}{\sigma'_v}}$</td><td>kg/cm^2, tsf</td></tr>
<tr><td rowspan="3">Skempton (1986)</td><td>$C_N = \frac{2}{1+\sigma'_v}$</td><td>Fine sands of medium relative density</td><td rowspan="3">kg/cm^2, tsf</td></tr>
<tr><td>$C_N = \frac{3}{2+\sigma'_v}$</td><td>Dense, coarse normally consolidated sands</td></tr>
<tr><td>$C_N = \frac{1.7}{.7+\sigma'_v}$</td><td>Overconsolidated, fine sands</td></tr>
</table>

4.5.2 Rod Length Correction Factor (C_R)

The energy from the SPT hammer is transmitted to the split spoon sampler and the soil through the drill rods. However, some of the energy is reflected back if the length of the rods is shorter than a critical length, which is defined as the length of the rod that weighs the same as the hammer (Schmertmann and Palacios 1979; Decourt 1990). The reflected energy reduces the energy available for the SPT and results in an increased measured N-value. Therefore, rod length correction factors have been proposed in the literature, are shown in Table 4.6.

Table 4.6. Rod length correction factors (Seed et al. 1984; Skempton 1986).

Rod Length	Rod Length Correction Factor (C_R)	
	Seed et al. (1984)	Skempton (1986)
< 10 ft (3 m)	.75	---
10 – 13 ft (3-4 m)	1.0	0.75
13 – 20 ft (4-6 m)	1.0	0.85
20 – 30 ft (6-10 m)	1.0	0.95
> 30 ft (> 10 m)	1.0	1.0

4.5.3 Borehole Diameter Correction Factor (C_B)

Boreholes that are larger than 4.5 inches in diameter can result in lower measured N-values due to the reduction in vertical effective stress at the sampling location. This factor is most significant for saturated sands.

4.5.4 Liner Correction Factor (C_S)

Split spoon liners are rarely used in practice in Rhode Island, which means that the inside diameter of the split spoon sampler increases from 1-3/8 inches to 1-1/2 inches. This results in a reduction in friction between the soil and the inside of the sampler and a reduction in the measured N-value.

4.6 Advantages and Disadvantages to the SPT

The Standard Penetration Test is one of the most commonly used in situ tests because of its simplicity and its considerable history of collected data. However, there are also limitations of the test that should be recognized. Table 4.7 and 4.8 presents a summary of advantages and disadvantages of the Standard Penetration Test as compiled from a number of researchers.

Table 4.7. Advantages of the Standard Penetration Test.

Advantages	Reference
Relatively quick and simple to perform	Kulhawy and Mayne (1990)
One procedure	Kulhawy and Mayne (1990)
Equipment and expertise for the test is widely available in the United States	Kulhawy and Mayne (1990)
Provides a representative soil sample	Kulhawy and Mayne (1990)
Provides useful index of relative strength and compressibility of the soil	NAVFAC DM 7.1 (1982)
Able to penetrate dense layers, gravel, and fill	NAVFAC DM 7.1 (1982)
Numerous case histories of soil liquefaction during past earthquakes are available with SPT N-values. The method based on this history can reflect actual soil behavior during earthquakes, which cannot be simulated in the laboratory.	Tokimatsu (1988)
The SPT is an in situ test that reflects soil density, soil fabric, stress and strain history effects, and horizontal effective stress, all of which are known to influence the liquefaction resistance but are difficult to obtain with undisturbed samples.	Tokimatsu (1988)

Table 4.8. Disadvantages of the Standard Penetration Test

Disadvantages	Reference
The SPT does not typically provide continuous data (e.g. 5 ft. intervals), therefore important data such as weak seams may be missed.	Kulhawy and Mayne (1990)
Limited applicability to cohesive soils, gravels, cobbles boulders	Kulhawy and Mayne (1990)
Somewhat slower than other sample methods due to sample retrieval	Kulhawy and Mayne (1990)
In addition to overburden pressure and relative density the SPT N-value is also a function of soil type, particle size, and age and stress history of deposit.	Kulhawy and Mayne (1990)
Due to considerable differences in apparatus and procedure, significant variability of measured penetration resistance can occur. The basic problems to consider are change in effective stress at the bottom of the borehole, dynamic energy reaching the sampler, sampler design, interval of impact, penetration resistance count	Tokimatsu (1988); Kovacs (1994)
Samples that are obtained from the SPT are disturbed.	

4.7 Correlations Between SPT and Soil Properties

Because the SPT is an indicator of soil consistency or density, the N-values have been correlated with a number of geotechnical engineering properties including relative density, soil friction angle, and undrained shear strength. **It is important to note, however, that some correlations require the raw N-values whereas others use the corrected $N_{1(60)}$ values.** When using any correlation, it is important to identify which blow count values are applicable. Given the inherent variability in the SPT method and results, it is also important to be aware that the blow counts are subject to uncertainty and should be used with judgment, especially when selecting engineering properties.

4.7.1 Relative Density

Relative density (D_r) is a measure of the relative compaction of a granular soil compared to its loosest and densest state. It applies to sands and gravels with less than 15% fines, and is defined as

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \text{ x } 100\% = \frac{\gamma_{d\max}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}} \text{ x } 100\% \tag{4.3}$$

where e = void ratio of the soil;

e_{min} = minimum void ratio;

e_{max} = maximum void ratio;

γ_d = dry unit weight of the soil;

γ_{dmin} = minimum dry unit weight; and

γ_{dmax} = maximum dry unit weight.

The maximum and minimum dry densities (or void ratios) of a given soil are determined in the laboratory. However, it can be difficult to obtain accurate and consistent values of e_{min} ($\gamma_{d\ min}$) that can be used in the field. Tables 4.9 and 4.10 and Figure 4.5 give correlations relating blow count to relative density.

Table 4.9. Relationship between relative density, standard penetration resistance, cone penetration resistance, and effective stress friction angle for sands and gravels (Meyerhoff 1956).

State of Packing	Relative Density	Standard Penetration Resistance (N)	Static Cone Resistance (q_c)	Angle of Internal Friction (ϕ')
	Percent	Blows / ft	Tsf or kgf/cm^2	Degrees
Very Loose	< 20	< 4	< 20	< 30
Loose	20 – 40	4 –10	20 – 40	30 – 35
Compact	40 – 60	10 –30	40 – 120	35 – 40
Dense	60 – 80	30 – 50	120 – 200	40 – 45
Very Dense	> 80	> 50	> 200	> 45

Table 4.10. Proposed correlations between relative density and SPT N-values (taken from McGregor and Duncan 1998).

Type of Soil	Relative Density	Parameters	Reference
Normally Consolidated Sands	$D_r = \sqrt{\dfrac{N}{1.7\left(10+\sigma_v'\right)}}$ (See Note)	σ_v' = vertical effective stress in psi	Gibbs and Holtz (1957); Holtz and Gibbs
Coarse Sands	$D_r = \sqrt{\dfrac{N}{0.773\sigma_v'+22}}$ for $\sigma_v' < 1560$ psf (75 kPa) $D_r = \sqrt{\dfrac{N}{0.193\sigma_v'+66}}$ for $\sigma_v' \geq 1560$ psf (75 kPa) (See Note)	σ_v' = vertical effective stress in kPa at location of test	Peck and Bazarra (1969)
Normally Consolidated Sands	$D_r = \sqrt{\dfrac{N_{60}}{a\sigma_v'+b}}$ If sand is overconsolidated, increase b by a factor C_f: $C_f = \dfrac{1+K_o}{1+2K_{onc}}$ where K_o = ratio of horizontal effective stress to vertical effective stress for overconsolidated sand ≈ $(1-\sin\phi')OCR^{\sin\phi'}$ Konc = ratio of effective horizontal stress to vertical stress for normally consolidated sand ≈ $(1-\sin\phi')$	N_{60} = blowcount corrected to 60% of the maximum free-fall energy a = 0.3 (mean value) b = 30 (mean value)	Skempton (1986)

Note: As originally proposed, this correlation used the uncorrected SPT N-values. However, hammers delivering 60% of the maximum free-fall energy have been the most commonly used hammers for standard penetration tests, and it seems likely that the data on which the correlation was based was obtained primarily from tests with such hammers. It therefore seems logical to use N_{60} with this correlation.

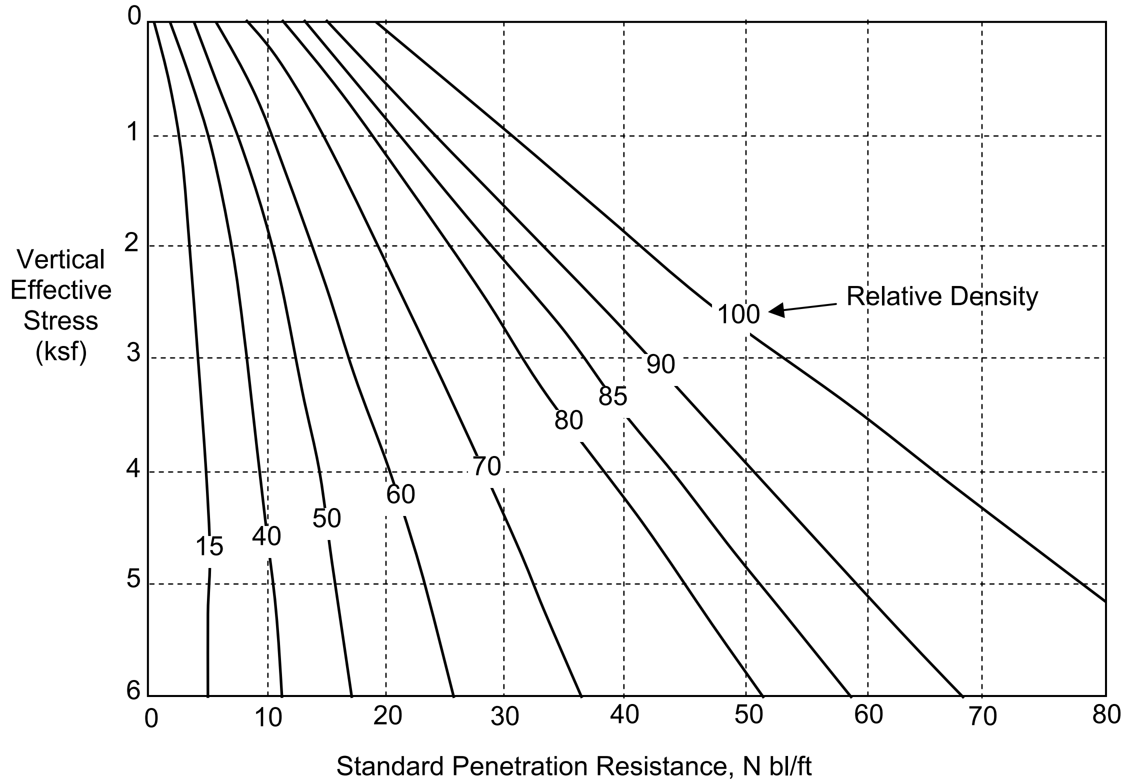


Figure 4.5. Correlation between relative density, vertical effective stress, and standard penetration resistance (after NAVFAC DM 7.2 1982).

4.7.2 Effective Stress Friction Angles of Sands

The effective stress friction angle quantifies the strength of a soil for granular material. The shear strength (τ) of soil is written as

$$\tau = \sigma' \tan\phi' + c' \quad (4.4)$$

where σ' = the effective stress acting on the soil;

ϕ' = the effective stress friction angle; and

c' = the effective stress cohesion.

The effective stress cohesion is generally considered to be zero for sands and gravels. Both ϕ’ and c’ are effective stress parameters because loading of sands and gravels is drained, meaning that the water can easily flow in or out of the soil during shear. Tables 4.11 and Figures 4.6 through 4.8 give correlations relating blow count to friction angle.

Table 4.11. Correlations between effective stress friction angle (ϕ’) of sands and silts and standard penetration resistance (as reported by McGregor and Duncan 1998).

Type of Soil	ϕ' (degrees)	Reference
Angular, well-grained soil particles	$\phi' = (12N)^{.5} + 25$	Dunham (1954)
Rounded, well-grained or angular, uniform sands	$\phi' = (12N)^{.5} + 20$	Dunham (1954)
Rounded, uniform-grained soil particles	$\phi' = (12N)^{.5} + 15$	Dunham (1954)
Sandy	$\phi' = (20N)^{.5} + 25$	Ohsaki (1959)
Granular	$\phi' = 3.5(N)^{.5} + 20$	Muromachi (1974)
Sandy	$\phi' = (15N)^{.5} + 15 \leq 45$ when $N > 5$	Japan Road Association (1990)
Sandy	$\phi' = (15.4(N_{1_{60}}))^{0.5} + 20$	Hatanaka and Uchida (1996)

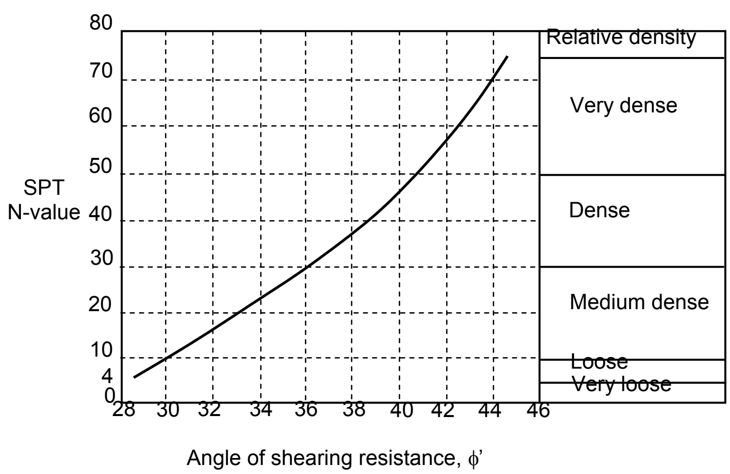


Figure 4.6. Estimation of the effective stress friction angle from standard penetration resistance (after Carter and Bentley 1991).

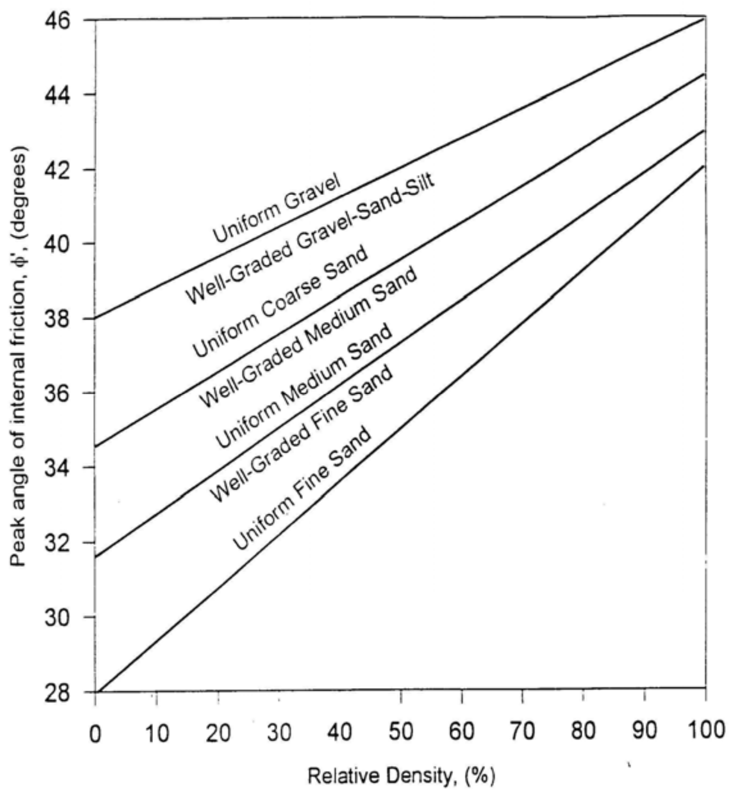


Figure 4.7. Relationship for angle of internal friction and relative density for different types of sand and gravel (Decourt 1990).

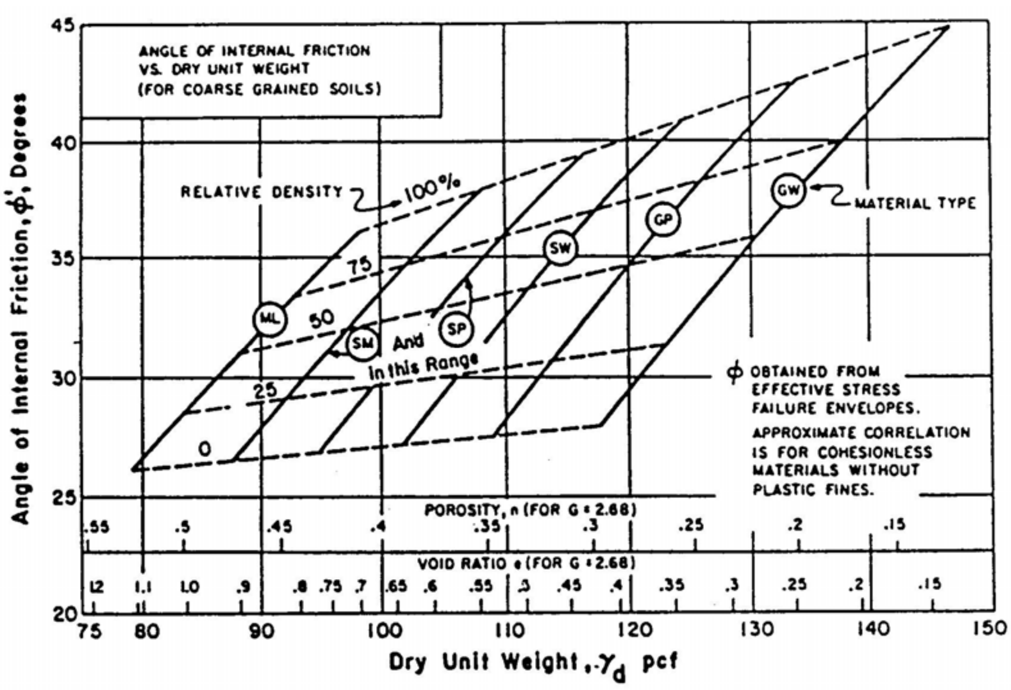


Figure 4.8. Correlation between effective stress friction angle, relative density, porosity, void ratio, and dry unit weight (NAVFAC DM 7.1 1982)

4.7.3 Undrained Shear Strength of Clays

The undrained shear strength, S_u, is applicable to clays in a short-term loading condition where water cannot flow in or out of the sample during shear, and instead excess pore pressures are developed. Table 4.12 and Figure 4.9 show published correlations between the standard penetration resistance and undrained shear strength of clays. The data upon which these relationships are based exhibits a large amount of scatter, particularly for tests in soft and sensitive clays (Terzaghi et al. 1996). These relationships should be used with caution and verified whenever possible with laboratory tests.

Table 4.12. Approximate values of undrained shear strength for cohesive soils based on the SPT blow count (Terzaghi et al. 1996).

Soil Consistency	SPT N	S_u (psf)	S_u (kPa)
Very Soft	< 4	< 250	< 12
Soft	2 – 4	250 – 500	12 – 25
Medium	4 - 8	500 – 1000	25 – 50
Stiff	8 – 15	1000 – 2000	50 – 100
Very Stiff	15 – 30	2000 – 4000	100 – 200
Hard	> 30	> 4000	> 200

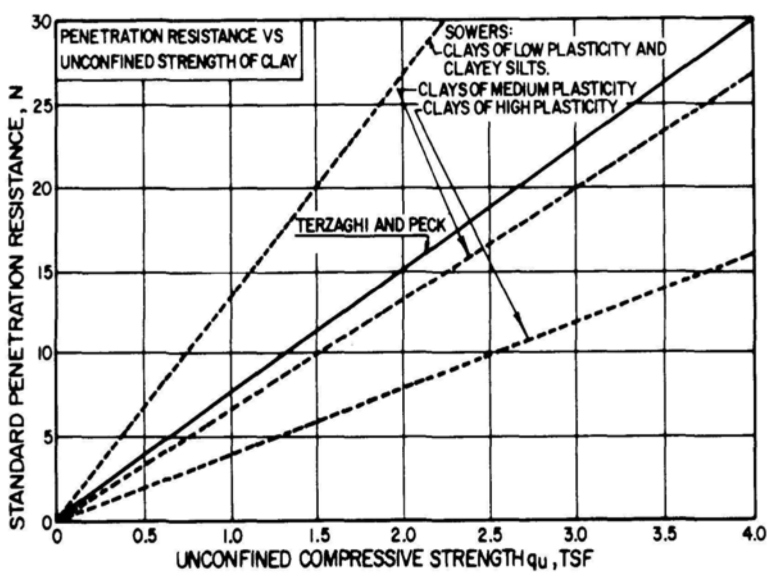


Figure 4.9. Relationship between standard penetration resistance and unconfined compressive strength (NAVFAC 1982)

5. Other In Situ Testing Methods

5.1 Introduction

In addition to the Standard Penetration Test, other *In situ* tests have been developed for the investigation of soil conditions or assessment of soil properties. These tests, including the cone penetration test (CPT), pressuremeter test, field vane test, and cross hole and down hole seismic surveys, are used to classify soil type and to measure a variety of properties such as shear strength and stiffness. In most cases, these are specialized tests that are not performed routinely and require expert operators to obtain accurate results.

This chapter summarizes the state-of-the-practice on the use and interpretation of data obtained from the cone penetration test, pressuremeter test, field vane test, and cross hole and down hole seismic surveys. These tests were chosen because they are the most common *in situ* tests that are used in Rhode Island aside from the SPT as shown in Table 5.1. For each test, a description of the equipment used and the recommended procedures is presented, followed by a discussion on the factors affecting the measured data, corrections to the data, and advantages and disadvantages of each test. Finally, correlations with engineering properties are presented.

Table 5.1. Selected RIDOT projects involving *in situ* tests.

Cone penetration test	
	Barrington Bridge Improvement
	Wellington Avenue Freight Rail Improvement Project
	Washington Bridge Project
	Manchester St. Power Plant
Pressuremeter	
	Providence Place Mall Ramp
	Barrington Bridge Improvement
Field Vane Test	
	Galilee Project
	Washington Bridge Project
Cross Hole Seismic Survey	
	Washington Bridge Project
	New Providence River Bridge

5.2 Cone Penetration Test (CPT)

The cone penetration test (CPT) is an *in situ* test in which an instrumented rod with a conical point is pushed into the soil. The most common measurements made during a CPT are the penetration resistance at the cone tip, the resistance along the side of the rod, and the excess pore pressure. However other specialized data is sometimes collected (e.g. shear wave velocity, electrical resistivity, video, etc.) for specific applications. Data collected from the CPT is used to classify soils, evaluate liquefaction potential, establish allowable bearing capacity, pile design, and the data may used to in conjunction with other *in situ* tests like the SPT to better characterize site conditions. The standard cone penetrometer has a diameter of 1.406 inches (35.7 mm), and a typical cross section is shown in Figure 5.1. Cone penetration tests can be performed from standard drill rigs, however it is more common to use specially designed cone trucks and tracked rigs that can operate in various terrain conditions. Examples are shown in Figure 5.2. The CPT is growing in popularity, especially for its use in soft, fine-grained soils and in fine to medium coarse sands. The test cannot be performed reliably in gravels and very stiff clays. A soil sample is not obtained from a CPT, however it still has several advantages over other in-situ testing methods. The CPT is fast, economical, and provides a continuous profile of soil stratigraphy and soil properties.

Figure 5.1 Typical cross-section of a Fugro electric cone penetrometer without pore pressure tranducers (Lunne et al. 1997)

Figure 5.2 Example of a cone truck and a track-mounted cone rig (http://www.conetec.com/body.htm)

5.2.1 Selected Publications on the Cone Penetrometer Test

Important references for the use and interpretation of the cone penetration test are listed below.

- American Society for Testing and Materials (2003). "Standard Test Method for Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soils (D 3441-94)," Annual Book of Standards, Vol. 4.08, ASTM, Philadelphia, 338-344.
- American Society for Testing and Materials (2003). "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils (D 5778-95)," Annual Book of Standards, Vol. 4.08, ASTM, Philadelphia, 570-587.
- Arman, A., Samtani, N., Castelli, R., and Munfakh, G. (1997). Geotechnical and Foundation Engineering Module 1 – Subsurface Investigations, FHWA-HI-97-021, 305 pp.
- Canadian Geotechnical Society (1985). Canadian Foundation Engineering Manual, 2nd Edition, Vancouver, 456 pp.
- Kulhawy, F. H., and Mayne, P.W. (1990). *Manual on Estimating Soil Properties for Foundation Design*, Electric Power Research Institute, 266 pp.
- Lunne, T., Robertson, P.K., and Powell, J.J.M. (1997). Cone Penetration Testing in Geotechnical Practice, Blackie Academic & Professional, 312 pp.
- Robertson, P. K., and Campanella, R.G., (1989). Guidelines for Geotechnical Design Using CPT and CPTU Data. Vancouver**,** 193 pp.

5.2.2 CPT Equipment and Procedures

The recommended procedure for performing the CPT is defined by the American Society for Testing and Materials specification D 3441 ("Standard Test for Deep, Quasi-Static, Cone and Friction-cone Penetration Tests of Soil") and D 5778 ("Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils"). Specification D 3441 describes how the tip resistance and sleeve friction are measured for both mechanical and electric cones, while D 5778 focuses on the use of electric piezocones to obtain penetration resistance and pore pressure measurements. An instrumented cone is attached to a string of steel rods and is pushed vertically into the ground at a constant rate of approximately 20 mm/sec. The equipment for pushing the cone consists of push rods, a thrust mechanism, and a reaction system. Trucks built specifically for the CPT include hydraulic jacking and reaction systems and are ballasted to a total dead weight of 15 tons or more (Lunne et al. 1997).

The standard cone penetrometer consists of a 60º pointed tip with a diameter of-1.406 in (35.7 mm) and a projected area of 1.55 in^2 (10 cm^2). The friction sleeve is the same diameter as the base of the cone and has a surface area of 23.2 in^2 (150-cm^2). Although this is the standard size, ASTM D 5778 allows for the use of a larger cone with a diameter of 1.72 in (43.7 mm), which has a projected tip area of 2.32 in^2 (15 cm^2) and a friction sleeve area of 31.0 in^2 (200 cm^2). Wires from the transducers in the cone are pre-threaded through the center of the rods, and the tip and side resistances are recorded throughout pushing of the cone. The measured point or tip resistance is designated q_c and the measured side or sleeve resistance is designated f_s.

In addition to the tip and sleeve resistance, other instruments can be installed in the cone penetrometer to make a variety of measurements. Inclinometers measure the verticality of the cone and can indicate when the cone is deflected excessively due to changes in soil stiffness or contact with obstructions. Excess and hydrostatic pore pressures can be measured at various locations on a piezocone (CPTU) to determine the soil type (fine grained vs. coarse grained) and to estimate the hydraulic conductivity of the soil. A seismic cone includes accelerometers that are used to measure the shear wave velocity of the surrounding soil. The data obtained with the CPT are presented on a profile of measured properties (e.g. tip resistance, sleeve resistance, pore pressure, etc.) with depth.

An example of a typical CPTU profile is shown on Figure 5.3. This profile was obtained from the Wellington Avenue Freight Rail Improvemennt Project in 2002. The variation of tip resistance, friction ratio, and pore pressure with depth are shown in the figure. The tip resistance decreases from a depth of approximately 25 feet to below approximately 50 feet with a slight increase in friction ratio. The magnitude of the values suggests a very soft soil deposit, which in this case is a thick layer of inorganic silt.

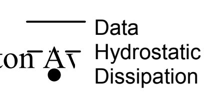


Figure 5.3. Typical CPT Profile obtained from the Wellington Av
Rail Improvement Project (DMJM Harris 2002)

Most cone penetration testing performed currently in Rhode Island uses piezocones (CPTU) to measure the pore pressure as well as the tip and sleeve resistance. The operating procedure for a CPTU is similar to the cone penetration test, however the preparation and instrumentation is more detailed. When performing a CPTU, careful consideration of the location and degree of saturation of the porous element must be made to ensure reliable measurements of pore water pressure. The measured excess pore pressures can either be positive or negative depending on the location of the measurement along the cone in addition to the volume change characteristics of the soil (i.e. dilatant or contractive). The standard location for the pore pressure measurement is directly behind the cone tip. This arrangement is called a Type 1 piezocone and is shown in Figure 5.4. Some piezocones measure pore pressures at the cone tip (Type 2, as shown in Figure 5.4), and there are also cones that measure pore pressures at up to three locations on the cone.

Figure 5.4. Location of porous element for Type 1 and Type 2 cones (Mayne et al. 2001).

5.2.3 Factors Affecting CPT Data

The cone penetration test is more automated and standardized than the SPT and there are fewer variables that can affect the measured tip resistance, sleeve friction, and pore pressures. However, some important factors have been identified by researchers and are described in Table 5.2.

Table 5.2 Factors that affect measured tip resistance (q_c), sleeve friction (f_s), and pore pressures in the cone penetration test (Lunne et al. 1997; Robertson and Campanella 1989).

Factors	Description
Pore pressure effects on tip and sleeve resistance ("Unequal Area Effect")	Pore pressures act on the exposed surfaces behind the cone tip and on the edges of the friction sleeve (see Figure 5.5). The tip resistance and sleeve resistance must be corrected for these pressures.
Filter location	The measured pore pressures depend greatly on whether the filter is located on the cone tip (u_1), directly behind the tip (u_2), or behind the friction sleeve (u_3).
Saturation of the pore pressure element	Unsaturated filters and pressure transducers will result in both inaccurate and delayed measurements of pore

	pressure.
Effect of axial load	Pore pressure measurements can be affected by axial load in the cone in some older versions of penetrometers. Most new cones that are commercially available do not have this problem.
Temperature effects	Changes in temperature can cause a shift in the load cell output at zero load
Inclination	The initial thrust direction should be within 2^{o} of vertical.

5.2.4 Recommended Procedures and Data Corrections to Measured CPT Data

The factors that are related to the characteristics of the equipment or procedures include the filter location, temperature effects, and inclination effects. Factors that can be corrected after the data are obtained are related to unequal area effects at the cone tip.

5.2.4.1 Filter Location

If only one pore pressure measurement is made on the cone, then in most cases it is recommended to place the porous element directly behind the cone tip (u_2). Lunne et al. (1997) present the following reasons for measuring pore pressures at the u_2 location:

- The filter is less susceptible to damage located behind the cone tip compared to on the cone tip.
- u_2 measurements are less influenced by compression of the cone tip during testing.
- u_2 measurements can be used directly to correct tip resistance for the unequal area effect.
- Pore pressures measured on the sleeve during a dissipation test (as opposed to on the cone tip) are less influenced by whether the rods are locked or not during the test.

5.2.4.2 Temperature

Most modern cones are equipped with temperature-compensated load cells. However, temperature effects can still be significant at small loads, such as when soft soils are encountered. These effects can be accounted for by taking zero readings before and after a CPT at the same temperature as the ground and by installing temperature sensors in the cone (Lunne et al. 1997).

5.2.4.3 Inclination

It is important that verticality of the cone be maintained in order to obtain accurate and representative measurements of the soil strata. Verticality of the cone can be easily checked using cones instrumented with slope sensors. If slope sensors are not used, Robertson and Campanella (1989) recommend that cone penetration tests can be performed up to a depth of 15 m without significant errors in the depth measurement providing obstructions do not exist.

5.2.4.4 Unequal Area Effect

The factors mentioned above must be taken into account in order to obtain accurate measurements of tip resistance, sleeve resistance, and pore pressure. An "unequal area effect" is caused by the inner geometry of the cone tip and filter, shown in Figure 5.5, and results in additional pore pressure acting on the ends of the friction sleeve and shoulder area behind the cone tip. The measured cone penetration resistance, q_c, is corrected for this effect using the following equation:

$$q_T = q_c + (1 - a_n)u_2 \tag{5.1}$$

where q_T = corrected cone penetration resistance;

q_c = measured cone penetration resistance;

u_2 = pore pressure measured on the sleeve just behind the cone tip; and

a_n = cone area ratio.

The cone area ratio is approximately equal to the ratio of the cross-sectional area of the load cell or shaft, A_n, divided by the projected cone area A_c, and can be determined experimentally (Lunne et al. 1997). Typical values of the cone area ratio range from 0.55 to 0.9. This effect is significant in soft to firm clays and silts in deep soundings where hydrostatic pressures are large. The effect is minimal in sands because the magnitude of the penetration resistance q_c is much greater than the measured pore pressure.

Figure 5.5 The effect of pore water pressure on measured values of tip resistance and sleeve friction (Lunne et al. 1997).

The sleeve resistance is also affected by pore pressures acting on the ends of the sleeve, and can be corrected if pore pressures are measured at both ends of the sleeve (u_2 and u_3 in Figure 5.5). Most commercial cones do not make u_3 measurements, and this correction is not usually performed. This introduces some uncertainty into the results of sleeve resistance, and is one reason why sleeve resistance measurements are not as reliable as measurements of tip resistance (Lunne et al. 1997).

5.2.5 Advantages and Disadvantages of the CPT

The cone penetration test is gaining popularity in the United States as an efficient *in situ* test for the estimation of soil properties. However, its use in the northeast has been limited mostly due to the limited availability of equipment and glacial soil deposits. A list of several advantages and disadvantages of the CPT is shown in Table 5.3.

Table 5.3 Advantages and disadvantages of the cone penetration test (Kulhawy and Mayne 1990).

Advantages	Disadvantages
• Faster than SPT in soft or loose soils • Penetration is interrupted only when additional rods need to be added • Continuous measurements of tip resistance, sleeve friction, and pore pressure are made • Inclinometers can be installed to monitor deflection of the penetrometer • Test personnel have a relatively minor influence on test result compared to SPT.	• No soil sample is recovered • Test results are unreliable in gravel soil and glacial till, where the penetrometer can be damaged • Mobilization of a special cone rig is expensive • Availability of companies able to perform CPTs in the northeast is limited.

5.2.6 Correlations Between CPT and Soil Properties

A number of correlations relating the results of CPT, SPT, and the engineering properties of soils have been developed. This section presents methods of soil classification from CPT data, estimation of relative density, effective stress friction angle, and undrained shear strength. Correlations relating SPT and CPT data are also presented. These correlations have not been developed for the soils found in Rhode Island, and should be used with engineering judgment.

5.2.6.1 Soil Classification

One of the major uses of the CPT is the classification of soil deposits. Because the CPT measures a continuous profile with depth, it is a much better tool than the SPT for identifying changes in soil strata and resolving thin layers such as sand and clay lenses. However, since the CPT cannot recover a soil sample, soil classification must be inferred from the measured tip resistance, sleeve friction, and pore pressure. Several methods have been developed to classify the soil using normalized values of tip resistance, sleeve friction, and pore pressure. Figure 5.6 shows a widely used chart for soil classification developed by Robertson (1990). In general, high values of tip resistance and low values of friction ratio indicate coarse grained materials, while low values of tip resistance, higher friction ratios, and significant excess pore pressures suggest the presence of fine grained soils. The classification chart shown in Figure 5.6 is based on a database of cone penetration tests performed in different geographical regions, and may not be representative of the soils found in Rhode Island. Charts such as these should always be calibrated with local soil conditions by obtaining samples from standard penetration tests, test pits, or other methods.

5.2.6.2 Relative Density

Relative density is a measure of the relative compaction of a granular soil compared to its loosest and densest state. It applies to sands and gravels with less than 15% fines, and is defined as

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \text{ x } 100\% = \frac{\gamma_{d\max}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}} \text{ x } 100\% \qquad (5.2)$$

where e = void ratio of the soil;

e_{min} = minimum void ratio;

e_{max} = maximum void ratio;

γ_d = dry unit weight of the soil;

γ_{dmin} = minimum dry unit weight; and

γ_{dmax} = maximum dry unit weight.

The maximum and minimum dry densities (and void ratios) of a given soil are determined in the laboratory.

Figure 5.6 Soil classification chart based on CPT or CPTU data (Lunne et al. 1997, after Robertson 1990).

The measured cone penetration resistance in coarse grained soils is strongly influenced by the density, the vertical and horizontal effective stresses, and the compressibility of the material. Figure 5.7 shows a correlation of cone penetration resistance, effective stress, and relative density for a medium dense sand. This correlation was developed from calibration chamber tests on Ticino sand in Italy.

Figure 5.7 Correlation relating cone penetration resistance, vertical effective stress, and relative density for normally consolidated sand (Lunne et al. 1997, after Baldi et al. 1986).

5.2.6.3 Effective Stress Friction Angle

The effective stress friction angle quantifies the strength of a soil for granular material. The shear strength (τ) of soil is written as:

$$\tau = \sigma' \tan\phi' + c' \qquad (5.3)$$

where σ' = the effective stress acting on the soil;

ϕ' = the effective stress friction angle; and c' = the effective stress cohesion.

The effective cohesion is generally considered to be zero for sands and gravels. Both ϕ' and c' are effective stress parameters because loading of sands and gravels is drained, meaning that the water can easily flow in or out of the soil during shear.

Various correlations have been developed that relate the cone penetration resistance to the effective stress friction angle, which provide a good estimate of the shear strength behavior of soils in advance or in lieu of a laboratory-testing program. Figure 5.8 shows a correlation of cone penetration resistance, vertical effective stress, and effective stress friction angle for uncemented quartz sands (Robertson and Campanella 1989).

Figure 5.8 Correlation between cone penetration resistance, vertical effective stress, and effective stress friction angle (Robertson and Campanella 1989).

5.2.6.4 Undrained Shear Strength of Clays

The undrained shear strength, S_u, is used to characterize the strength of clays when water cannot flow in or out of the sample during shear, and instead excess pore pressures are developed. Undrained shear strength plays an important role in determining the short-term stability of foundations, slopes, and embankments consisting of fine grained soils. Many relationships have been developed relating cone penetration resistance to undrained shear strength using both theoretical (Yu and Mitchell 1998) and empirical approaches (Lunne et al. 1997). For an initial estimate of undrained shear strength, the following empirical relationship can be used:

$$S_u = \frac{(q_t - \sigma_{vo})}{N_{kt}} \tag{5.4}$$

where q_c = measured total cone penetration resistance;

σ_{vo} = total *in situ* vertical stress; and

N_{kt} = empirical cone factor.

Values of N_{kt} vary with the plasticity index of the soil and can be estimated using Figure 5.9. It should be emphasized that site-specific correlations should be developed whenever possible for a more accurate estimate of undrained shear strength in Rhode Island.

5.2.6.5 Relationship between CPT and SPT

The standard penetration test is the most widely used *in situ* test in Rhode Island, and in many cases engineers prefer to convert CPT resistance to equivalent SPT N-values. Figure 5.10 shows the relationship between q_c, N, and median grain size.

Figure 5.9 Cone factor, N_{kt}, as a function of plasticity index
(Lunne et al. 1997, after Aas et al. 1986).

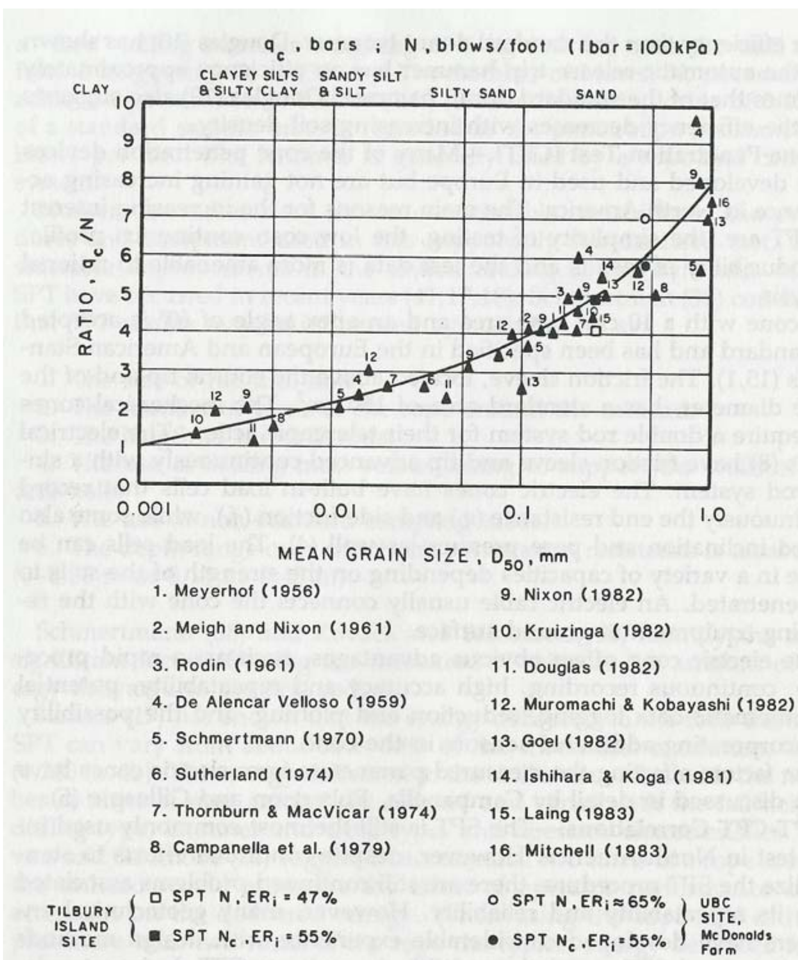


Figure 5.10 Correlation between cone penetration resistance, standard penetration resistance, and mean grain size (Robertson et al. 1983).

5.3 Pressuremeter Test (PMT)

The pressuremeter is an *in situ* testing device in which an inflatable cylinder is placed in a borehole and inflated radially into the soil. The pressure applied to the borehole wall and the volume change of the pressuremeter are recorded and used to obtain a soil modulus, shear strength (either drained or undrained), and horizontal stress conditions.

Kögler developed the first pressuremeter in 1933 and L.F. Menard developed and refined the pre-bore pressuremeter at the University of Illinois in 1956 (Mair and Wood 1987). Several configurations of the PMT have been developed and refined since then. The pressuremeter can be a useful tool for the evaluation of soil properties and the design of foundations. It is especially useful for characterizing soils that are difficult to sample or test using conventional geotechnical methods, such as glacial till, soft clays and silts, soft rock, dense granular soils, frozen soil, layered sands and soil containing gas in the pores (Canadian Geotechnical Society 1985). The installation and calibration of the pressuremeter and the interpretation of the test results is complex and requires considerable experience to obtain accurate soils data.

5.3.1 Selected Publications on the Pressuremeter Test

Important references for the use and interpretation of the pressuremeter test are listed below.

- American Society for Testing and Materials (2003). “Test Method for Pressuremeter Testing in Soils (D 4719-87),” Annual Book of Standards, Vol. 4.08, ASTM, Philadelphia, 861-868.
- Arman, A., Samtani, N., Castelli, R., and Munfakh, G. (1997). Geotechnical and Foundation Engineering Module 1 – Subsurface Investigations, FHWA-HI-97-021, 305 pp.
- Briaud, J. L. (1989). “The pressuremeter test for highway applications.” *Report FHWA-IP-89-008*, Federal Highway Administration, Washington, D.C., 148 pp.

- Canadian Geotechnical Society (1985). Canadian Foundation Engineering Manual, 2nd Edition, Vancouver, 456 pp.
- Kulhawy, F. H., and Mayne, P.W. (1990). *Manual on Estimating Soil Properties for Foundation Design*, Electric Power Research Institute, 266 pp.
- Naval Facilities Engineering Command (1982). Soil Mechanics Design Manual 7.1. DM-7.1.

5.3.2 Pressuremeter Test Equipment and Procedures

There are two types of pressuremeters that are currently used in practice. The Menard pressuremeter (MPM) is used in a pre-drilled borehole, usually after pushing and removing a Shelby Tube. The self boring pressuremeter (SBPM) was developed to reduce the soil disturbance caused by traditional drilling and sampling techniques. The SBPM is pushed into the soil, and a cutting head/auger system at the end of the device excavates the soil and flushes it up through the center of the pressuremeter with drilling fluid. Diagrams of the Menard and self boring pressuremeters are shown in Figure 5.11.

Figure 5.11 Schematic Diagrams of the Menard Pressuremeter and the Self-boring Pressuremeter (AASHTO 1988).

The recommended procedure for performing the PMT using the Menard pressuremeter is defined by the American Society for Testing Materials specification D 4719 entitled "Test Method for Pressuremeter Testing in Soils." The pressuremeter is placed in a pre-drilled borehole and is expanded, usually in equal pressure increments. It is extremely important that the walls of the borehole be as clean as possible (i.e. undisturbed). The volume change of the pressuremeter for each increment of pressure is plotted as shown in Figure 5.12. This data is used to determine three characteristic pressures that are used to interpret the results of the test (Kulhawy and Mayne 1990):

p_o– the pressure at which the pressuremeter begins expansion into undisturbed soil

p_f – a yield pressure where the soil behavior changes from pseudo-elastic to plastic.

p_L – the limit pressure at which complete yielding or plastic behavior of the soil occurs.

These pressures are used to estimate the horizontal stress state, modulus, and shear strength of the soil.

Figure 5.12. Typical volume change behavior of a pressuremeter during a PMT (Canadian Geotechnical Society 1985).

5.3.3 Factors Affecting Pressuremeter Test Data

Given the complexity of the calibration, installation, and performance of the pressuremeter test, there are a number of factors that can significantly affect the test results. Important factors that have been identified from the literature are shown in Table 5.4.

Table 5.4. Factors and variables that affect the results of the Menard pressuremeter test and the self-boring pressuremeter test (Mair and Wood 1987; Baguelin et al. 1978).

Factors	Description
Calibration	Evaluates corrections required for pressure and volume losses, temperature, and hydrostatic pressure. Requires considerable experience to accurately calibrate the results.
Pressure Loss	This is the pressure required to expand to probe in air. It is a function of the rigidity of the probe walls, and is a significant factor in soft soils.
Volume or Radius Changes	This is due to the compressibility of the probe and the tubing. This factor can be significant when testing stiffer soils and weak rocks.
Method of Installation for MPMT	This is a significant factor. Each site has to be considered individually; no single method of installation is always suitable.
Cutter position for SBPT	This needs to be optimized for each soil stratum.
Size of Cutting Shoe for SBPT	This should be identical to the diameter of the pressuremeter.
Rate of Penetration for SBPT	The force applied to the drilling rods, rate of cutter rotation, and pumping of flushing fluid to remove the cuttings affects the rate of penetration for the SBPT.
Excessive Vibration for	This has to be prevented from being transferred down

the SBPT	the drill rods otherwise significant soil disturbance occurs.
Inherent Heterogeneity in the Soil	May affect the limit pressure differently than the modulus.

5.3.4 Corrections to Pressuremeter Data

The raw data must be corrected for pressure and volume losses, temperature, and the hydrostatic pressure at the testing depth. These parameters are evaluated during the calibration process and applied to the pressuremeter data to obtain the corrected values. Applying appropriate corrections to the raw data requires experience and should only be performed by qualified personnel. Figure 5.12 shows corrected data of volume vs. pressure applied to the borehole wall.

5.3.5 Advantages and Disadvantages to the Pressuremeter Test

The pressuremeter test is becoming a useful in-situ test in soils that are difficult to sample and test with traditional techniques. Advantages and disadvantages of this test are described in Table 5.5.

Table 5.5 Advantages and disadvantages of the pressuremeter test
(Kulhawy and Mayne 1990; Canadian Geotechnical Society 1985; Arman et al. 1997).

Advantages	Disadvantages
• Can estimate horizontal stress state • Theoretical foundation for determining soil properties • A larger zone of soil is tested than with most other *in situ* tests • Excellent tool for designing pile foundations for lateral load conditions.	• Complicated calibration and testing procedures • Requires experts to conduct the test and interpret the results • Time consuming and expensive • Does not obtain a soil sample.

5.3.6 Correlations Between the Pressuremeter Test and Soil Properties

Three characteristic pressures are obtained from the volume-pressure relationship shown in Figure 5.12. These are the lift-off pressure where the probe expands into undisturbed soil (p_o), a yield pressure where the soil behavior changes from pseudo-elastic to plastic (p_f), and the limit pressure at which complete yielding of the soil occurs (p_L). These pressures are used to estimate the horizontal stress state, modulus, and shear strength of the soil.

5.3.6.1 Horizontal Stress State

It is often assumed that the lift-off pressure, p_o, is equal to the total horizontal stress in the ground. Therefore, the at-rest lateral earth pressure coefficient can be estimated by

$$K_o = \frac{\sigma_h'}{\sigma_v'} = \frac{p_o - u}{\sigma_v'} \qquad (5.5)$$

where σ_h' = horizontal effective stress;
σ_v' = vertical effective stress; and
u = hydrostatic water pressure.

It should be noted that the lift-off pressure is extremely sensitive to sample disturbance, and some references (e.g. Canadian Geotechnical Society 1985) specifically state that p_o should not be considered equal to the total horizontal stress.

5.3.6.2 Modulus

The slope of the volume-pressure relationship in the pseudo-elastic region (between p_o and p_f) is used to obtain an elastic modulus that is often used for the analysis of lateral loads on piles and drilled shafts. The equivalent Young's modulus, E_{PMT}, can be written as

$$E_{PMT} = 2(1+\nu)V(\Delta P / \Delta V) \qquad (5.6)$$

where ν = poisson's ratio (between 0.33 and 0.5), V = current volume of the probe, ΔP/ΔV = slope in the pseudo-elastic region (Mayne et al. 2001).

5.3.6.3 Undrained Shear Strength of Cohesive Soils

The undrained shear strength, S_u, can be estimated from the limit stress, p_L. Using cylindrical cavity expansion theory (Baguelin et al. 1978), S_u can be evaluated by

$$S_u = (p_L - p_o)/ N_p \tag{5.7}$$

where

$N_p = 1 + \ln (E_{PMT} / 3 S_u)$

5.3.6.4 Effective Stress Friction Angle

The interpretation of pressuremeter tests in sands is complicated by the fact that the tests are drained and therefore volume changes in the sand around the expanding cavity are able to occur freely. Because of this fact, the estimation of the effective stress friction angle is quite complicated. One should refer to Mair and Wood (1987) for a detailed description on how to obtain estimates of effective stress friction angles from the pressuremeter test.

5.4 Field Vane Test (FVT)

The vane shear test (VST) or the field vane (FVT) is a very useful tool for measuring the *in situ* undrained shear strength of soft, saturated, cohesive soils. The test involves inserting a four-bladed vane in undisturbed soil at the bottom of a borehole and rotating the vane until the soil fails. The torque required to rotate the vane is measured, and the undrained shear strength can be calculated knowing the maximum torque and the geometry of the vane. The field vane is the most common method of determining the undrained shear strength in soft to firm clays, and is not applicable in cohesionless soils. Both the peak and remolded shear strengths can be measured during a field vane test.

5.4.1 Selected Publications on the Field Vane Test

Important references for the use and interpretation of the field vane test are listed below.

- American Society for Testing and Materials (2003). "Standard Test Method for Field Vane Shear Test in Cohesive Soil (D 2573-94)," Annual Book of Standards, Vol. 4.08, ASTM, Philadelphia, 239-241.
- Arman, A., Samtani, N., Castelli, R., and Munfakh, G. (1997). Geotechnical and Foundation Engineering Module 1 – Subsurface Investigations, FHWA-HI-97-021, 305 pp.
- Canadian Geotechnical Society (1985). Canadian Foundation Engineering Manual, 2nd Edition, Vancouver, 456 pp.
- Chandler, R. J. (1988). "The In-Situ Measurement of the Undrained Shear Strength of Clays Using the Field Vane," *Vane Shear Strength Testing in Soils: Field and Laboratory Studies, ASTM STP 1014*, A. F. Richards, Ed., ASTM, Philadelphia, 13-44.
- Kulhawy, F. H., and Mayne, P.W. (1990). *Manual on Estimating Soil Properties for Foundation Design*, Electric Power Research Institute, 266 pp.

5.4.2 Field Vane Testing Equipment and Procedures

The recommended procedure for performing a FVT is defined by the American Society for Testing and Materials specification D 2573 ("Standard Test Method for Field Vane Shear Test in Cohesive Soil"). The test involves pushing a four-bladed vane into undisturbed soil at depth and rotating it until the soil fails. The torque required to fail the soil is measured and converted to the undrained shear strength based on the geometry of the vane. There are currently three types of field vanes used in the United States (AASHTO 1988):

1. Standard field vane with torque wrench – this is the simplest arrangement for the FVT. The blade is inserted into the soil at the bottom of a borehole. A torque wrench is used to rotate the vane from the ground surface and only gross shear strength data is obtained.

2. Precision torque held assembly (“Acker” model) – this is used in cased borings and can accurately maintain a constant rate of rotation. The variation of torque with the angle of rotation can be measured throughout the test. This is shown in Figure 5.13.

3. Self-contained portable vane (“Geonor”, SGI vane borer) – this vane can provide its own cased boring.

A variety of different sized vanes are used depending on the stiffness of the soil. The standard vane geometry has a height-to-width ratio of 2:1 with a blade height of 130 mm, diameter of 65 mm, and a blade thickness of 2 mm. This geometry is shown in Figure 5.14.

Figure 5.13 Photograph and schematic view of Field Vane Shear device (http://civcal.media.hku.hk/airport/investigation/fieldwork/vane/_hidden/vane1.htm; NAVFAC 1982).

The top of the vane should be pushed to a depth of insertion of at least 4 times the borehole diameter when performing the FVT (Mayne et al. 2001). Within five minutes of insertion, rotation should be made at a constant rate of 6° per minute (0.1^{o} per second) with measurements of torque taken frequently. Once failure has been reached, the vane should be rotated 10-12 revolutions to completely remold the soil around the vane. After another period of rest, the FVT is repeated to obtain the residual shear strength of the soil.

Figure 5.14. Geometry of the straight and tapered field vanes (ASTM D 2573).

Assuming a uniform shear stress distribution along the top and bottom of the blades and a 2:1 height-to-diameter ratio, the field vane undrained shear strength (S_{uv}) is calculated by

$$S_{uv} = \frac{6T_{\max}}{7\pi D^3} \tag{5.8}$$

where T_{max} is the maximum torque measured during the test and D is the diameter of the vane (Chandler 1988). It should be noted that S_{uv} must be corrected to obtain values of undrained shear strength that can be used for stability analyses.

5.4.3 Factors Affecting Field Vane Test Data

The procedure, equipment, and methods vary for the VST, therefore several factors and variables affect the testing results. Table 5.6 describes the most significant factors.

Table 5.6 Factors and variables that affect the results of the field vane test (Canadian Geotechnical Society 1985; Mayne et al. 2001)

Factors	Description

Friction along rod of the vane	The friction along the rods must be subtracted from the measured torque to obtain an accurate assessment of shear strength. This can be done using casing or by performing a second series of tests adjacent to the FVT with only the rods and subtracting the measured torque of the rods from the FVT. However, the use of casing is recommended.
Blade Thickness	Blade thickness should not exceed 5 percent of the vane diameter.
Vane Blade	Vane blade should have a height-to-diameter ratio of 2.
Time Correction	Since the undrained shear strength test result is time dependent the vane test results must be corrected for time effects.
Highly Plastic Clays	Field vane greatly overestimates the undrained strength of many highly plastic clays especially if they contain roots, shells, sand lenses, and varves.
Other Effects	The field vane test is affected by rotation of principal planes during shear, dimensions of vane and failure cylinder in the soil, rate of rotation, and disturbance during insertion.

5.4.4 Corrections to Measured Field Vane Data

The measured field vane strength, S_{uv}, must be corrected prior to use in stability analyses involving embankments, excavations, and foundations in soft, cohesive soils. The field vane test overestimates the undrained shear strength, S_u, for highly plastic clays, and Bjerrum (1973) developed the following correction:

$$S_u = \mu \; S_{uv} \tag{5.9}$$

where; the correction factor, μ, is dependent on the plasticity index, PI, of the soil, as shown in Figure 5.15. This empirical correction was developed from back-analyses of failures in soft clays where field vane data was available.

Figure 5.15. Field vane correction factor as a function of plasticity index (Bjerrum 1973).

Chandler (1988) recommends that the correction factor be determined by

$$\mu = 1.05 - b(PI)^{0.5} \tag{5.10}$$

where $b = 0.015 + 0.0075 \log t_f$; and

t_f = time to failure in minutes.

For embankments on soft clays, the time to failure is assumed to be approximately 1 week (10,000 minutes) to approximate the time of construction.

5.4.5 Advantages and Disadvantages to the Field Vane Test

The field vane test is a useful in-situ test for determining the undrained shear strength of soft to stiff cohesive soils. Table 5.7 presents a list of advantages and disadvantages described by various authorities.

Table 5.7Advantages and disadvantages of the field vane test (Kulhawy and Mayne 1990).

Advantages	Disadvantages
• Rapid and economical test • Reproducible results in homogeneous soils • Equipment and test are simple • Long history of use in engineering practice • Inexpensive method for measuring clay sensitivity.	• Limited to soft to stiff cohesive soils • Field vane shear strength must be corrected • Results can be affected by anisotropic soils, sand lenses, shells, and seams.

5.4.6 Correlations Between Field Vane Test Data and Soil Properties

The field vane test yields a direct measure of the undrained shear strength when the data is properly corrected. Correlations are not used to relate the field vane data with other engineering properties.

5.5 Wave Propagation Seismic Survey

In addition to the *in situ* tests described above, there are a variety of geophysical tests that can provide fast and economical supplementary information about subsurface conditions at a site. These include resistivity tests, ground penetrating radar, seismic reflection studies, cross hole and down hole seismic surveys, and seismic cone penetration tests. In general, geophysical techniques are useful for the

- Determination of the stratigraphy of a site;
- Identification of abrupt changes in soil or rock formations;
- Measurement of dynamic properties *in situ;*
- Identification of cavities in karst regions; and
- Identification of underground obstructions (Arman et al. 1997).

Of these methods, cross hole and down hole seismic surveys have been performed more often in Rhode Island, and these techniques will be described further in this section. Seismic surveying involves imparting mechanical wave energy into soil or rock surrounding a borehole and measuring the travel times of the waves from the energy source to detectors in the same borehole or adjacent boreholes (AASHTO 1988). The seismic tests typically measure shear wave velocity, which is related to the shear modulus of the soil and is not influenced by the presence of the groundwater table. Shear wave velocity and shear modulus are properties that are useful in evaluating the dynamic response of the soil from machine vibrations or earthquakes. These tests require considerable expertise in the installation, performance, and interpretation of the results and should only be conducted by qualified personnel.

5.5.1 Selected References on Cross Hole and Down Hole Seismic Testing

Important references for the use and interpretation of the cross hole and down hole seismic tests are listed below.

- American Society for Testing and Materials (2003). “Standard Test Methods for Crosshole Seismic Testing (D 4428),” Annual Book of Standards, Vol. 4.08, ASTM, Philadelphia, 636-645.

- American Association of State Highway and Transportation Officials (1998). Manual on Subsurface Investigations, Washington, D. C., 391 pp.
- Arman, A., Samtani, N., Castelli, R., and Munfakh, G. (1997). Geotechnical and Foundation Engineering Module 1 – Subsurface Investigations, FHWA-HI-97-021, 305 pp.
- Stokoe, K. H. and Woods, R. D. (1972). "*In Situ* Shear Wave Velocity by Cross-Hole Method," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 98, No. SM5, 443-460.

5.5.2 Cross Hole and Down Hole Seismic Test Equipment and Procedures

The recommended procedure for performing a cross hole seismic test is defined by the American Society for Testing and Materials specification D 4428 ("Standard Test Methods for Crosshole Seismic Testing"). The cross hole test involves two or three boreholes installed in a line at a spacing of 10 to 15 ft (3 to 4.5 m), as shown in Figure 5.16. A seismic energy source is placed at different depths in the borehole, and seismic receivers (geophones) are placed in the other boreholes at the same elevation. The source is activated and the travel times of the waves (compression or shear waves, depending on the type of source) between the boreholes are recorded. When the source is activated, it triggers the recording device and the receivers. Knowing the travel times and the distance between the boreholes, the compression or shear wave velocities can be determined.

The energy source is usually a down hole hammer that clamps to the side of the borehole and is powered by a hydraulic pump. Because it is important to have good contact between the hammer and geophones with the surrounding soil, the borehole should be cased and grouted into place.

The receivers are typically geophones or accelerometers. Each receiver needs to be able to measure waves in the vertical and two horizontal directions at right angles to each other. The recording device is usually a spectrum analyzer or digital oscilloscope.

The down hole test is similar to the cross hole test except that the source energy is generated at the ground surface (usually by striking a block of wood that is held under the wheel of a truck) and only one bore hole is used for the receiver.

5.5.3 Factors Affecting Cross Hole and Down Hole Seismic Test Data

Several factors and variables have been identified that can significantly affect the results of the cross hole and down hole seismic tests. These factors are described in Table 5.8.

Figure 5.16 Diagram of a typical cross hole seismic test (Arman et al. 1997)

Table 5.8 Factors and variables that affect the results of the cross hole and down hole tests (Arman et al. 1997; Mayne et al. 2001).

Factors	Description
Borehole Size	Boreholes for the receivers and the source should be kept as small as possible.
Borehole Preparation	Boreholes should be PVC cased and annular voids filled with sand or low-density grout.
Elevations of Energy Source and Detectors	The elevations of the energy source and detectors must be known and well controlled so that the depths and thicknesses of subsurface layers, as well as distances between energy source and detector, can be accurately determined.
Cross-Hole Tests	It is desirable to use a generating source rich in shear and low in compression to increase the amplitude of the shear wave and help in delineating its arrival time.

5.5.4 Advantages and Disadvantages of the Cross Hole and Down Hole Tests

Table 5.9 presents a list of advantages and disadvantages described by various authorities.

Table 5.9 Advantages and disadvantages of the cross hole and down hole seismic tests (Kulhawy and Mayne 1990).

Advantages	Disadvantages
• Non-destructive test • Fast and economical • Theoretical basis for interpretation • Applicable for both soil and rock • Indication of average soil properties is provided rather than properties at localized areas • The larger volume of tested material is important for improvement measures that produce a non-homogenous soil mass.	• No samples are obtained • Assumed model for analysis of layered soils. • Results are affected by cemented layers or inclusions.

5.5.5 Correlations Between Cross Hole and Down Hole Tests and Soil Properties

The cross hole and down hole seismic tests yield direct measurements of the shear wave (or compressional wave) velocities. The shear modulus can be obtained from measurements of the shear wave velocity. However, correlations are not widely used to relate the seismic data with other engineering properties.

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